ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)

Bridge No. 08080R, M.P. 60.42
State Project No. 301-099

Submitted by:
Stantec Consulting Services Inc.

December, 2010

Abstract

Stantec Consulting Services Inc. (Stantec) has been assigned by the Connecticut Department of Transportation (CTDOT) to inspect and analyze the Devon Bridge (CTDOT Bridge No. 08080R), carrying Amtrak's busiest rail corridor over the Housatonic River in southwestern Connecticut. Our inspection revealed that the bridge is in poor condition. Stantec has developed six alternatives to address the poor condition of the bridge, which range from low investment, short term repairs to a full structure replacement. Ultimately, the selection of which alternative to implement will be CTDOT's with recommendations from Stantec based on funding availability, annualized costs and impacts to rail and river traffic. At this time, based on the current condition of the bridge, Stantec recommends a full structure replacement.

The bridge is an integral component of the Metro-North New Haven Commuter Line and Amtrak Northeast Corridor Line, carrying an estimated 6,300 passengers daily across the Housatonic River. Located at M.P. 60.42 of the New Haven Line, the bridge spans both the border between the City of Milford and Town of Stratford as well as the border between New Haven and Fairfield County. The bridge was constructed in 1905, underwent a major rehabilitation in 1990, and is subject to on-going maintenance efforts. Based on its age, condition, serviceability, and current maintenance requirements, the CTDOT has begun to develop a program to ensure the existing crossing of the Housatonic River is maintained for rail traffic in the future.

The bridge is comprised of parallel twin structures, with each structure carrying two active tracks. The northerly structure carries Tracks 1 and 3, while the southerly structure supports Tracks 2 and 4. Twin single leaf bascule spans over the navigable channel of the Housatonic River. The bridge is located immediately to the north, and parallel to, the Moses Wheeler Bridge which carries Interstate 95 roughly 70 feet above the Housatonic River. It is noted that this bridge is currently in the process of being replaced by CTDOT with a scheduled completion date of 2015. Three High Towers associated with the bridge carry Metro-North Railroad high voltage feeder lines, catenary jumper lines, and catenary wires along with United Illuminating high voltage power lines over the Housatonic River.

Structurally, the bridge consists of a seven (7) span, open deck structure with an overall length of 1,067 feet. Spans are numbered 1 through 7 consecutively from west to east, and have overall span lengths of 146.33 feet, 110.42 feet, 34.37 feet, 110.00 feet, 222.58 feet, 222.42 feet and 220.58 feet, respectively. Span 1 consists of gusset-plated Warren trusses with a floorbeam and stringer flooring system. Span 2 consists of riveted deck girders. Span 3 consists of deck girders supporting end and mid-span floorbeams with stringers. The girders also support the rolling lift during bridge opening events. Span 4 consists of a "Scherzer Rolling Lift" through truss with floorbeams and stringers. Truss members are connected with gusset plates and rivets. Spans 5, 6 and 7 consist of pin-connected Baltimore trusses and a floorbeam and stringer flooring system. Built-up compression members and multiple flat plate eyebar tension members form the truss. See Figure 2.1.

Stantec completed an in-depth inspection, analyzed the existing structure, developed alternatives for the repair, rehabilitation, and replacement of the bridge, and prepared life cycle cost analyses associated with each of the alternatives. The intent of this report is to summarize the findings of these efforts, and to provide CTDOT with a comprehensive report to allow for management decisions regarding the future allocation of funds relative to the repairs and long term options associated with the bridge.

In general, the bridge was found to be in poor condition due to deficiencies noted in the superstructure and substructure during the in-depth inspection. The controlling "As-inspected" load rating analysis for the bridge is E-50 (Normal) based on the standard design loading of a Cooper E-80 Train, located at Truss 3 of Span 7 (Member U8-M9), supporting Tracks 2 and 4. Compared with the "As-built" controlling rating of E-62 (Normal), it is clear that structural deterioration of the main load carrying members has occurred. However, the Cooper E-80 design train is somewhat heavier than the commuter rail cars that use the bridge on a routine basis, and further load ratings will be required to assess the bridge's capacity to support the commuter cars. Analysis of the three High Towers indicates that many of the tower members are overstressed, and that a large imbalance of longitudinal loads from the cables is present. A preliminary analysis indicates that the current high towers become overstressed at approximately 50-60 mph based on current design codes.

Stantec also investigated various other aspects related to the bridge, as they contribute to its functionality and performance. These included analyzing the hydraulic and scour performance of the bridge in existing and future configurations, reviewing possible environmental and permitting requirements that would be necessary for the various rehabilitation or replacement alternatives, reviewing subsurface conditions, identifying potential historic impacts associated with modifying or replacing the existing bridge, evaluating marine navigational requirements, identifying potential utilities impacts, and finally, establishing both short- and long-term railroad operational and maintenance requirements.

Based on the information collected and evaluated, Stantec then developed conceptual rehabilitation and replacement alternatives that partially or fully address deficiencies that were found. The alternatives were developed based on three service life horizons: 5-7 year, 25-year, and 75-year. The alternatives were segregated as such to allow for decisions regarding the future allocation of funds relative to the repairs and long term options associated with the Devon Bridge and the Northeast Corridor trains that the bridge serves.

The alternatives developed are as follows:

- **Alternative I – Short Term Repair.** This alternative consists of performing localized repairs or replacement of deteriorated members to increase the useful life of the bridge by 5 to 7 years.
- **Alternative II – Rehabilitation.** This alternative involves performing major repairs to both the super- and substructure, replacement of major structural members and systems and construction of additional items to upgrade the useful life of the bridge to approximately 25 years. The existing high towers will be replaced with new monotube towers.
- **Alternative IIIa – Partial Superstructure Replacement.** This alternative involves replacing Spans 5, 6, and 7 of the superstructure while rehabilitating Spans 1, 2, 3 and 4, and using the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years. The existing high towers will be replaced with new monotube towers.
- **Alternative IIIb – Complete Superstructure Replacement.** This alternative involves replacing the entire superstructure and using the majority of the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule. The substructure units that support the movable span types other than the Scherzer rolling lift will require new substructure units. The existing high towers will be replaced with new monotube towers.
- **Alternative IVa – Full Replacement with Trusses.** This alternative involves replacing both the superstructure and the substructure to increase the useful life of the bridge to 75+ years. The superstructure consists of thru trusses similar to existing. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule. The existing high towers will be replaced with new monotube towers.
- **Alternative IVb – Full Replacement with Deck Girders.** This alternative involves replacing both the superstructure and the substructure to increase the useful life of the bridge to 75+ years. The superstructure consists of a deck girder system. New movable bridge types investigated with

this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule. The existing high towers will be replaced with new monotube towers.

Capital cost estimates were developed for each of the six alternatives, with costs developed in year 2010 dollars based on unit costs including labor, material, and equipment expenses for each item. Miscellaneous Items (Mobilization, Minor Items, etc.) and construction cost contingency were added to this cost on a percentage basis of 40% and 20% respectively, as directed by CTDOT staff. Costs for Signals & Communication, Catenary, and Track & Rail work were developed by CTDOT staff and added to this subtotal to develop a Total Construction Cost for each alternative.

Additional items were added on a percentage basis as directed by CTDOT staff for Force Account work (40%), Engineering (15%), Incidental Expenses (20%). Finally, an overall project contingency of 15% was added to develop the total project cost. The total project cost was then projected out to the anticipated midpoint of construction at a rate of 6% per year to develop the anticipated future project cost.

A summary of the results of the cost analysis are presented in the following table:

Preliminary Cost Estimate*	
Alternative I	$3,000,000**
Alternative II	$280,000,000
Alternative IIIa	$580,000,000
Alternative IIIb	$660,000,000
Alternative IVa	$840,000,000
Alternative IVb	$790,000,000

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access costs provided by CTDOT staff
**Assumes long term rehabilitation to follow within 5-7 years

A life-cycle cost analysis (LCCA) was developed to provide a financial metric to assist in the evaluation of the alternatives presented in this report. Alternative I was not evaluated, as the work of this alternative will be performed to bridge the gap between the current conditions and a larger scale rehabilitation effort. A summary of the results of the cost analysis are presented in the following table:

Life Cycle Cost Analysis	
Alternative	Equivalent Uniform Annual Cost (EUAC)
Alternative II	$14,000,000
Alternative IIIa	$14,000,000
Alternative IIIb	$12,100,000
Alternative IVa	$14,900,000
Alternative IVb	$14,000,000

Based on a review the advantages and disadvantages of the six alternatives, Alterantive IVb – Full Replacement with Deck Girders appears to be the most advantageous rehabilitation option. While not the least expensive option in terms of both initial cost as well as annualized cost, this option will provide an entirely new and reliable structure that will be designed entirely in accordance with current codes and standards. The cost annualized cost differential between the next most reasonable alternative (Alternative IIIb) is approximately 15%, and the initial costs of each are comparable as well.

The work proposed with Alternative IVb will:

- Be composed of an entirely new structure with a 75+ year life span;
- Address deficiencies of pin and eyebar connections;
- Facilitate more conventional superstructure erection techniques (ie crane picks);
- Be seismically adequate;
- Be designed for scour;
- Be designed for vessel impact;
- Improve reliability of the structure and movable span;
- Facilitate future higher speeds through the corridor;
- Provide for minimal future maintenance;
- Allow for increased navigation channel width;
- Correct deficiencies in the High Towers; and
- Be consistent with current efforts to upgrade the Northeast Corridor to a high speed rail facility.

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I. Executive Summary

A. Introduction

Stantec Consulting Services Inc. (Stantec) has conducted an Engineering, Feasibility and Economic Analysis of the Metro-North Railroad Bridge over the Housatonic River (Devon Bridge), Bridge No. 08080R, M.P. 60.42. The bridge is located across the border between the City of Milford and Town of Stratford, as well as the border between New Haven and Fairfield County.

The bridge is an integral component of the Metro-North New Haven Commuter Line and Amtrak Northeast Corridor Line, carrying an estimated 6,300 passengers on approximately 150 trains daily across the Housatonic River. Constructed in 1905, the existing bridge underwent a major rehabilitation in 1990 and is subject to on-going maintenance efforts. Based on its age, condition, serviceability, and current maintenance requirements, the Connecticut Department of Transportation (CTDOT) has begun to develop a program to ensure the existing crossing of the Housatonic River is maintained for rail traffic in the future.

In support of this program, Stantec completed an in-depth inspection, analyzed the existing bridge structure, developed alternatives for the repair, rehabilitation, and replacement of the bridge, and prepared life cycle cost analyses associated with each of the alternatives. The intent of this report is to summarize the findings of these efforts, and to provide CTDOT with a comprehensive report to allow for executive decisions regarding the future allocation of funds relative to the short and long term options associated with the bridge.

B. Methodology

Stantec conducted a comprehensive evaluation of the existing bridge prior to commencing the alternatives analysis. This evaluation centered on determining the condition and structural capacity of the existing bridge and three associated high towers. Additional investigations were also conducted prior to developing alternatives analysis.

To assess the condition of the bridge, Stantec performed in-depth structural, mechanical and electrical inspections of the existing bridge and high tower components in accordance with the CTDOT Bridge Inspection Manual. Prior to commencing the field inspection, Stantec reviewed previous inspection, load rating, and various other reports to become familiar with anticipated field conditions, inspection procedures, and access requirements. Stantec, assisted by A. DiCesare Associates and Garg Consulting Services, then performed the field inspection primarily between May 4, 2009 and June 25, 2009, with additional days of inspection and verification of findings through September 2009. Inspection of the mechanical and electrical components required observation of the movable span in operation, which occurred on June 14, 2009. During the field inspection, inspectors also took measurements of critical structural components and associated losses due to deterioration. In addition, non-destructive testing of the bridge truss pins and an analysis of the existing paint system were also performed during this time period by specialty subconsultants.

Stantec conducted load rating analyses of each of the bridge spans based on field conditions noted during the in-depth inspection. The load rating analyses were performed in accordance with the 2007 Manual for Railway Engineering published by the American Railway Engineering and Maintenance-of-Way Association (AREMA). The structural model used to perform the structural analysis was based on the original design and shop drawings, subsequent rehabilitation drawings, and actual field measurements. Information from these sources was used to identify member section properties and material properties. All primary load carrying members within each of the seven spans were evaluated for both a baseline "As built" condition (assuming no member deterioration) as well as an "As inspected" condition (including member deterioration).

Members evaluated during the load rating included trusses, floorbeams, stringers, girders, and truss pins. Evaluations used a standard Cooper E-80 live loading pattern as a basis for the analyses. Each member was evaluated for the aforementioned "As built" and "As inspected" conditions at Normal, Maximum, and Fatigue allowable stress levels for axial, moment and shear forces. In addition, Stantec also conducted a seismic analysis of the existing bridge to assess its vulnerability to seismic events.

Each of the three high towers was evaluated based on the load combinations specified in the 2007 National Electrical Safety Code. A structural model was developed using a lattice frame structure. Tower geometry, member size and member configuration for the as-built tower models of the outer towers were based on available plans on file for the high towers of the 1912 Stamford-New Haven Electrification project by the New York, New Haven & Hartford Railroad Company. The plans on file did not have specific information for the Devon Bridge location, but the Devon Bridge outer towers were identical to the other locations shown on the plans. No plans were available for the middle tower or for the tower bridges; these were modeled using as-inspected field measurements.

Stantec also investigated various other aspects related to the bridge, as they contribute to its functionality, performance, and presence on the line. These included analyzing the hydraulic and scour performance of the bridge in existing and future configurations, reviewing possible environmental and permitting requirements that would be necessary for the various rehabilitation or replacement alternatives, reviewing subsurface conditions, identifying potential historic impacts associated with modifying or replacing the existing bridge, evaluating marine navigational requirements, identifying potential utilities impacts, and finally, establishing both short- and long-term railroad operational and maintenance requirements.

Based on the information collected and evaluated, Stantec then developed conceptual rehabilitation and replacement alternatives that partially or fully address deficiencies that were found. The alternatives were developed based on three service life horizons: 5-7 year, 25-year, and 75-year. The alternatives were segregated as such to allow for decisions regarding the future allocation of funds relative to the repairs and long term options associated with the Devon Bridge and the Northeast Corridor trains that the bridge serves.

C. Description of Bridge

The Devon Bridge carries the Metro-North New Haven Commuter Line and the Amtrak Northeast Corridor Line over the Housatonic River at the Stratford/Milford town line. Located at milepost 60.42 along New Haven's Commuter line, the bridge is immediately west of the Waterbury Branch spur. The bridge is comprised of parallel twin structures, with each structure carrying two active tracks. The northerly structure carries Tracks 1 and 3, while the southerly structure supports Tracks 2 and 4. Twin single leaf bascule spans over the Housatonic allow for ship navigation up river. The bridge is located immediately to the north, and parallel to, the Moses Wheeler Bridge which carries Interstate 95 roughly 70 feet over the Housatonic River. It is noted that the I-95 bridge is currently in the process of being replaced by CTDOT with a scheduled completion date of 2015.

The bridge consists of a seven (7) span, open structure with an overall length of 1,067 feet. Spans are numbered 1 through 7 consecutively from west to east, and have overall span lengths of 146.33 feet, 110.42 feet, 34.37 feet, 110.00 feet, 222.58 feet, 222.42 feet and 220.58 feet, respectively. Span 1 consists of gusset-plated Warren trusses with a floorbeam and stringer flooring system. Span 2 consists of riveted deck girders. Span 3 consists of deck girders supporting end and mid-span floorbeams with stringers. The girders also support the rolling lift during bridge opening events. Span 4 consists of a "Scherzer Rolling Lift" through truss with floorbeams and stringers. Truss members are connected with gusset plates and rivets. Spans 5, 6 and 7 consist of pin-connected Baltimore trusses and a floorbeam and stringer flooring system. Built-up compression members and multiple flat plate eyebar tension members form the truss. See Figures G-1 and G-2.

Photo I-1: Aerial View Looking South

The bridge is supported by two gravity type masonry abutments founded on spread footings, with six intermediate piers constructed of masonry with concrete foundations founded well below the mudline. The abutments are labeled Abutments 1 and 2, from west to east. The piers are labeled Piers 1 through 6, again from west to east. A timber pile supported fender system protects Piers 3 and 4, which are adjacent to the navigation channel below movable bascule Span 4.

Photo I-2: View from West High Tower looking East

The Operator's House contains the bridge controls as well as electrical components associated with the movable span. The house is located between Piers 2 and 3 on the south side of the bridge. Access is provided by walking across the bridge along Track 4, and then climbing a series of steel ladders and stairs to the Operator's House level. The structure is built using conventional wood framing supported by steel sub-framing. An open steel deck wraps around the west, south, and east sides of the house.

High towers (integral to catenary towers 862 and 863) carry Metro-North Railroad high voltage feeder lines, catenary jumper lines, and catenary wires along with United Illuminating high voltage power lines over the Housatonic River. The middle tower does not carry catenary wires and does not have an official catenary number designation; for the purposes of this report, it is referred to as Catenary 862A. Catenary 862 is located west of the bridge in Milford, Catenary 862A is located on the bridge on pier 3 just east of the bascule span, and Catenary 863 is located to the east in Stratford. The structures were built in 1912. All of the towers rise approximately 198.50 feet above ground level and are connected at three levels by truss bridges. The lower, middle, and upper bridges are 24.50 feet, 99.67 feet, and 170.50 feet above ground level. The tower legs are comprised of steel angles with cover plates braced together with smaller angles and channel sections. The towers are supported on reinforced concrete foundations. The bottom 1½ panels of Catenary 862A are encased in concrete.

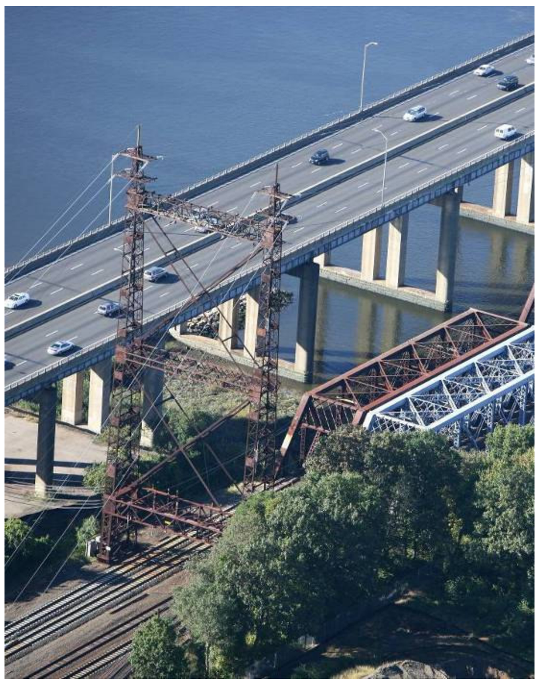


Photo I-3: High Tower at East Approach

Metro-North operates approximately 55 scheduled trains eastbound and 52 scheduled trains westbound during normal weekdays over the bridge. Amtrak operates approximately 22 scheduled trains in both the eastbound and westbound directions during normal weekdays. The bridge also serves freight traffic (approximately 3 per day).

The bridge opening log indicates an average of 95 bridge openings per year over the past two years including monthly test openings. Aside from scheduled monthly test openings, the opening of the bridge is initiated by a request from marine vessel operators.

D. Summary of Inspection Findings

The structure was inspected between May 4, 2009 and June 25, 2009, with additional days of inspection and verification of findings through September 2009. Inspection of the mechanical and electrical components required observation of the movable span in operation, which occurred on June 14, 2009. The inspection report is contained in 13 volumes as follows:

VOLUME I	SUMMARY REPORT
VOLUME II	FIELD NOTES FOR SPAN NOS. 1, 2, 3, & 4
VOLUME III	FIELD NOTES FOR SPAN NO. 5
VOLUME IV	FIELD NOTES FOR SPAN NO. 6
VOLUME V	FIELD NOTES FOR SPAN NO. 7
VOLUME VI	FIELD NOTES FOR SUBSTRUCTURE UNITS, OPERATOR'S HOUSE, DAILY LOGS, PAINT TESTING, ULTRASONIC TESTING OF PINS, & TRACK INSPECTION
VOLUME VII	UNDERWATER INSPECTION
VOLUME VIII	MECHANICAL AND ELECTRICAL INSPECTION
VOLUME IX	HIGH TOWER INSPECTION
VOLUME X	LOAD RATING ANALYSIS FOR SPAN NOS. 1, 2, & 3
VOLUME XI	LOAD RATING ANALYSIS FOR SPAN NOS. 4, 5, 6, & 7, AND SEISMIC ANALYSIS
VOLUME XII	LOAD RATING ANALYSIS FOR HIGH TOWERS (PART 1)
VOLUME XIII	LOAD RATING ANALYSIS FOR HIGH TOWERS (PART 2)

In general, the bridge was found to be in poor condition due to deficiencies noted in the superstructure and substructure during the in-depth inspection.

Superstructure

The superstructure steel exhibits corrosion, pitting loss of section, general section loss, and impacted rust at various locations throughout. Driving the poor rating of the bridge are conditions of two components of the primary members: the stringers primarily under Track 1, and the pinned truss connections and associated eye bars of Spans 5, 6, and 7. The stringers typically exhibit loss of section at the top flange in the form of pitting losses at the top plate and/or edge losses of the flange members. In addition, pitting type web losses were also noted in the stringers.

The structural steel throughout balance of the bridge exhibits various degrees of deterioration. The primary steel members generally exhibit corrosion, pitting, section loss, and impacted rust at various degrees of severity. The connections between stringers and floorbeams exhibit moderate to heavy impacted rust, and section loss is exhibited at the bottom of the connection angles. The paint system on the bridge was found to have completely failed on a majority of the structural steel. The bridge pins continue to exhibit rotational movement at many locations throughout the bridge, which has resulted in losses in the connecting eye bars due to friction. The results of an ultrasonic testing examination of the pins did not reveal the presence of any crack-like indicators in any of the truss pins examined.

Substructure

The poor rating of the substructure components is due to exposure and deterioration of pier footings at Piers 2, 3, and 4. Although no undermining was noted, previous underwater inspection reports, along with the current inspection results, indicate active scour (aggregation and degradation) is occurring at all piers. Above the footings, the masonry pier stems, as well as the abutment stems are in satisfactory condition, with minor loss of mortar and hairline cracks noted in the masonry.

Track

The track system consists of continuous welded rail fastened to wooden ties with Pandrol 'e' Clip style fasteners, tie plates, and spikes. The ties are blocked and strapped together throughout. Square cut miter rail joints with outside rider rails are present at the movable span.

The rail and fasteners were noted to be in good condition. No fasteners were noted to be missing, and all tie plates had two spikes fastening it to the ties. The timber ties were generally in good condition. Isolated ties were noted at various locations with splitting and rot for up to the full width of the tie, with the majority of these noted in Spans 6 and 7.

Miter rails are located at the movable span rail joints. These joints are in fair condition. No missing bolts were noted. The rails exhibit no end batter at these joints. All headblocks exhibited moderate to excessive wear.

Mechanical

The bridge machinery was found to be in generally good condition. The spans operate infrequently (approximately 95 times annually) for marine traffic; the lack of operations has reduced wear but also appeared to have reduced the frequency of machinery lubrication.

Span Drive Machinery – During operation of the north span the north and south intermediate gearsets were alternately driving the span, possibly due to unequal total backlash between the north and south side gearing. The racks and rack pinions were heavily worn from many years of service. The machinery supports and shafts were all in satisfactory condition with paint failure and light corrosion typical for their age. The enclosed differential speed reducers were in good condition with little indication of wear, and bearings were in good condition with little measurable wear, but require more frequent lubrication. Five of the eight brakes did not either fully set or fully release.

Span Lock Machinery – The span lock machinery operated smoothly and quietly. However, the system was heavily worn, corroded and no longer serves its original function of holding the span in the seated position. Each leaf has a single open gearset driven by a gearmotor, with satisfactory alignment and light to moderate wear. The machinery supports, shafts, cranks, links, bearings, and locks are all unpainted with corrosion. The span lock hooks also exhibited excessive clearance and as such did not lock the span down in the seated position. It was reported by Metro-North personnel that if the hooks are adjusted to hold the spans down they get stuck when attempting to unlock the spans.

Span Support Machinery – The horizontal track plates and curved tread plates were in good condition, but were unlubricated with light corrosion. The live load shoes were unpainted with heavy corrosion, with loose anchor bolts. The live load shoes were all in firm contact with the spans seated. The north leaf south live load shoe pumped under train traffic and had a thin shim plate added over the pier shoe. The centering devices consist of vertical bent plates attached to the live load shoes on the extreme north and south sides of each span. The plates were unpainted with corrosion, and there was clearance at the north side of both spans and hard contact at the south side of both spans. The air buffers were non-functional; however with the modern control system installed on the spans the air buffers are no longer necessary as the spans seat automatically.

Electrical

The electrical system was found to be operational and in good serviceable condition. The system is of previous-generation design, and at the time of the inspection, one operating system was out of service. All motors, starters, contactors, breakers, wiring and terminations are in good condition. The external breaker handles must be held in place in the breaker closed position or the vibration from the trains can cause the external handle to trip the breakers. The motors are in generally good condition, with minor maintenance required to brushes and resistors. Toe lock motors and brakes are also in good condition. Lastly, the navigation lighting was in good condition.

Operator House

The Operator's House is in serviceable condition, with minor cosmetic deficiencies. The Bridge Operator on duty at the time of our inspection noted that the heating and cooling systems were adequate, but that the ventilation in the bathroom is inadequate when the toilet is incinerating the waste.

The steel framing for the Operator's House is in poor condition. The most significant deterioration is located along the base of the eastern columns where up to 80% section losses are noted at outboard angle legs. The western column bases have been repaired and are in better condition. Other deterioration includes heavy pitting, groove section losses and perforations in gussets, web plates and angles at angle and lattice girder bracing.

High Towers

The high towers are in poor condition. At the end towers (862 and 863), all legs have section loss up to ½ inch deep on the interior sides of the angles along the edges of vertical gusset plates at the splices, and cover plate loss up to 90%, typical at splices with thin shim plates. The horizontal bracing members at lower elevations have extensive section loss at isolated locations. The section loss is typically at the member connections. The gusset plates at the horizontal member connection typically have section loss up to ⅜ inch deep along the top of the horizontal member. There are isolated gusset plates that have section loss around all sides of the horizontal member. On the middle tower (862A), the only sizeable section loss to the leg angles is on the tower legs at the base. The bracing and lacing bars also have section loss at the concrete encasement interface with up to 100% section loss on isolated members. The anchor bolts are in poor condition, with section losses up to nearly 50% at some locations.

E. Analyses

Load Rating Analysis

The controlling "As-inspected" load rating analysis for the bridge is E-50 (Normal) based on a Cooper E-80 Train Loading. The controlling load rating is governed by Truss 3 of Span 7 (member U8-M9), which supports Tracks 2 and 4. Compared with the "As-built" controlling rating of E-62 (Normal), it is clear that structural deterioration of the main load carrying members has occurred.

High Towers

The analysis on file with the Connecticut Department of Transportation from October 2001 considers the catenary towers to be structurally adequate. However, both the current "As-built" and "As-inspected" load rating of these towers indicates that many of the tower members are significantly overstressed. This is attributable to several factors, including the evolution of design codes since the structure was designed as well as the addition of ground wire extensions at the top of the towers. Further, the analysis indicates a large imbalance of longitudinal loads from the cables, creating a constant moment force on the towers.

F. Description of Alternatives

Based on the information collected and evaluated, Stantec then developed conceptual rehabilitation and replacement alternatives that partially or fully address deficiencies that were found. The alternatives were developed based on three service life horizons: 5-7 year, 25-year, and 75-year. The alternatives were segregated as such to allow for decisions regarding the future allocation of funds relative to the repairs and long term options associated with the Devon Bridge and the Northeast Corridor trains that the bridge serves. The six alternatives were developed for the repair/rehabilitation/replacement of the bridge as follows:

- **Alternative I – Short Term Repair.** This alternative consists of performing minor repairs or replacement of deteriorated members to increase the useful life of the bridge by 5 to 7 years.
- **Alternative II – Rehabilitation.** This alternative involves performing major repairs to both the super- and substructure, replacement of major structural members and systems and construction of additional items to upgrade the useful life of the bridge to approximately 25 years.
- **Alternative IIIa – Partial Superstructure Replacement.** This alternative involves replacing Spans 5, 6, and 7 of the superstructure while rehabilitating Spans 1, 2, 3 and 4, and using the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years.
- **Alternative IIIb – Complete Superstructure Replacement.** This alternative involves replacing the entire superstructure and using the majority of the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule. The substructure units that support the movable span types other than the Scherzer rolling lift will require new substructure units.
- **Alternative IVa – Full Replacement with Trusses.** This alternative involves replacing both the superstructure and the substructure to establish the useful life of the bridge to 75+ years and ensure seismic compliance. The superstructure consists of thru trusses similar to existing. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule.
- **Alternative IVb – Full Replacement with Deck Girders.** This alternative involves replacing both the superstructure and the substructure to establish the useful life of the bridge to 75+ years. The superstructure consists of a deck girder system. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule.

For Alternatives III and IV, the existing high towers carrying high voltage feeder lines will be replaced with new monotube towers.

<u>Alternative I – Short Term Repair</u>

This alternative consists of performing localized repairs to the structural steel superstructure, minor repairs to the substructure, and minor repairs and/or upgrades to the mechanical and electrical components. A number of steel stringers exhibit edge and pitting losses at the flanges at all truss spans which contribute to the reduction in load capacity. At several locations, localized repairs to truss members will be performed. In addition, a number of secondary members (lateral bracing) will be repaired as required. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking.

This alternative also includes repairs to the mechanical and electrical systems. While some of these items fall into the category of maintenance, they are worthy of including in a Short Term repair plan. In addition to lubricating all moving components, any loose fasteners will be tightened and failed or cracked welds repaired. Brake shoe contact will be adjusted at all locations. The tips of the south rack pinion of the north bascule leaf will be ground down to eliminate tooth bottoming.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair treads, railings and railing posts will also be repaired or replaced. An new ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

Estimated Construction Cost = $3,000,000*

*assumes MNR staff will perform repairs

Implementation of Alternative 1 assumes that long term rehabilitation to follow within 5-7years, and annual inspection of the bridge will continue.

Alternative II – Rehabilitation

This alternative consists of performing substantial repairs to the structural steel superstructure, repairs to the substructure, and a major upgrade to the mechanical and electrical components. Many of the built up stringers, primarily below Track 1 (Stringers 3 and 4), will be replaced with wide flange members. Floorbeams will be repaired as required at truss spans. Repairs to remaining truss and secondary members considered in Alternative I will be completed. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking. A scour monitoring system will be installed at Piers 2, 3, 4, and 5.

An on-site standby generator will be installed to provide power to the bridge control system and Operator's House during power outages. All other repairs noted during Alternative I will be completed as well.

The major components of the mechanical system will be replaced, including the racks and rack pinions, various drive gearsets, span locks, and live load shoes and centering devices. In addition, the air buffers will be removed.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair treads, railings and railing posts will also be repaired or replaced. An adequate ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

All structural steel, including Spans 1 through 7, machinery, and Operator's House support, will be blast cleaned and painted.

The fender system will be replaced with new.

The existing high towers will be replaced with new monotubes.

Estimated Project Cost at Midpoint of Construction = $280,000,000*

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access cost provided by CTDOT staff

Alternative IIIa – Partial Superstructure Replacement

This alternative consists of replacing Spans 5, 6 and 7 with new truss structures, completing a rehabilitation of the Spans 1, 2, 3, and 4 to address deficiencies noted in the structural steel, performing repairs to and modifying the substructure, and completing a major upgrade of the mechanical and electrical components. Many of the stringers (in particular those below Track 1) will be replaced with new and floorbeams will be repaired at Spans 1, 3, and 4. The deck girders of Span 2 will be repaired as in Alternative I. The new spans at Spans 5, 6, and 7 would be similar to existing, with the exception that the truss members would be rolled steel sections and would be connected using bolted gusset plates. The floor system would be a floorbeam and stringer configuration similar to existing. Repairs to remaining truss and secondary members at Spans 1 through 4 considered in Alternative I will be completed. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking. A scour monitoring system will be installed at Piers 2, 3, 4, and 5.

The masonry piers will be retrofitted to improve seismic performance.

An on-site standby generator will be installed to provide power to the bridge control system and Operator's House during power outages. All other repairs noted during Alternative I will be completed as well.

The major components of the mechanical system will be replaced, including the racks and rack pinions, various drive gearsets, span locks, and live load shoes and centering devices. In addition, the air buffers will be removed.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair tread, railing and railing posts will also be repaired or replaced. An adequate ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

All existing structural steel, including Spans 1 through 4, machinery, and Operator's House support, will be blast cleaned and painted. New structural steel will be painted or galvanized.

The fender system will be replaced with new.

The existing high towers will be replaced with new.

Estimated Construction Cost at Midpoint of Construction = $580,000,000*

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access cost provided by CTDOT staff

<u>Alternative IIIb – Complete Superstructure Replacement</u>

This alternative consists of replacing all superstructure spans truss structures, performing repairs to and modifying the substructure, and replacing all of the mechanical and electrical components with new. If the movable span is replaced with a vertical lift, new piers will be required to support the lift towers and span. Span 1 will be replaced in kind with a new truss. The new spans at Spans 5, 6, and 7 would also be similar to existing, with the exception that the truss members would be rolled steel sections and would be connected using bolted gusset plates. The floor system for Spans 1, 5, 6, and 7 would be a floorbeam and stringer configuration similar to existing.

Several movable span options are available with this alternative; the configuration of the movable span will dictate the structure type for Spans 2, 3 and 4. Available options for the movable span include:

- Rolling Bascule (Scherzer Rolling Lift)
- Heel Trunnion Bascule
- Vertical Lift

However, the existing pier layout naturally favors reusing the existing movable span type of a Rolling Bascule in terms of cost and constructability when considering a superstructure replacement. As such, only the Rolling Bascule was progressed and developed as a movable span type for this alternative.

The existing masonry piers will be retrofitted to improve seismic performance.

The existing high towers will be replaced with new.

Estimated Construction Cost = $660,000,000*

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access cost provided by CTDOT staff

<u>Alternative IVa – Full Replacement with Trusses</u>

This alternative involves the complete replacement of the existing bridge on the existing alignment. The bridge will consist of spans with piers located to avoid the existing piers and maintain the existing navigation channel. This translates to five spans of approximately 217 feet, 200 feet, 217 feet, 217 feet, and 217 feet in length. The spans will consist of through trusses similar to that of the existing bridge in shape. However the truss members will consist of rolled steel shapes connected with bolted gusset plates. The floor system would be a floorbeam and stringer configuration similar to existing.

The substructures will be constructed of reinforced concrete abutments and piers supported by deep foundations, either reinforced concrete drilled shafts or driven piles. The deep foundations would bear on or be socketed into bedrock.

Due to the long span at the navigation channel, the only practical configuration for the movable span is a vertical lift.

The existing high towers will be replaced with new.

Estimated Construction Cost = $840,000,000*

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access cost provided by CTDOT staff

Alternative IVb – Full Replacement with Deck Girders

This alternative involves the complete replacement of the existing bridge on the existing alignment. The bridge will consist of spans with piers located to avoid the existing piers, maintain the existing navigation channel, and maximize the spans of the deck girder system. This translates to eight spans of approximately 114 feet, 114 feet, 200 feet, 124 feet, 124 feet, 124 feet, and 126 feet in length. The spans will consist of deck girders similar to those found on Span 2 of the existing bridge. The movable span will consist of a through truss due to the span length.

The substructures will be constructed of reinforced concrete abutments and piers supported by reinforced concrete drilled shafts socketed into bedrock.

Due to the long span at the navigation channel, the only practical configuration for the movable span is a vertical lift.

The existing high towers will be replaced with new.

Estimated Construction Cost = $790,000,000*

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access cost provided by CTDOT staff

G. Summary of Alternatives

A review of the results of the in-depth inspection report and as-built and as-inspected load ratings reveal that the Devon Bridge has undergone significant deterioration since its construction in 1905, and is due for major rehabilitation or replacement. The six alternatives that were investigated as part of this project could be developed into feasible rehabilitation projects that will address the deficiencies of the bridge in varying degrees.

Capital cost estimates were developed for each of the six alternatives, with costs developed in year 2010 dollars based on unit costs including labor, material, and equipment expenses for each item. Miscellaneous Items (Mobilization, Minor Items, etc.) and construction cost contingency were added to this cost on a percentage basis of 40% and 20% respectively, as directed by CTDOT staff. Costs for Signals & Communication, Catenary, and Track & Rail work were developed by CTDOT staff and added to this subtotal to develop a Total Construction Cost for each alternative.

Additional items were added on a percentage basis as directed by CTDOT staff for Force Account work (40%), Engineering (15%), Incidental Expenses (20%). Finally, an overall project contingency of 15% was added to develop the total project cost. The total project cost was then projected out to the anticipated midpoint of construction at a rate of 6% per year to develop the anticipated future project cost.

A summary of the results of the cost analysis are presented in the following table:

Preliminary Cost Estimate*	
Alternative I	$3,000,000*
Alternative II	$280,000,000
Alternative IIIa	$580,000,000
Alternative IIIb	$660,000,000
Alternative IVa	$840,000,000
Alternative IVb	$790,000,000

*Excludes Force Account, Catenary, Signals & Communications, Traction Power, & Station Access; Assumes long term rehabilitation to follow within 5-7 years

A life-cycle cost analysis (LCCA) was developed to provide a financial metric to assist in the evaluation of the alternatives presented in this report. Alternative I was not evaluated, as the work of this alternative will be performed to bridge the gap between the current conditions and a larger scale rehabilitation effort. A summary of the results of the cost analysis are presented in the following table:

Life Cycle Cost Analysis	
Alternative	Equivalent Uniform Annual Cost (EUAC)
Alternative II	$14,000,000
Alternative IIIa	$14,000,000
Alternative IIIb	$12,100,000
Alternative IVa	$14,900,000
Alternative IVb	$14,000,000

A qualitative relative comparison of each alternative is listed in the following table:

Qualitative Comparison of Alternatives						
Alternative:	I	II	IIIa	IIIb	IVa	IVb
Construction Duration	+		-	-	-	-
Constructability	-	-			+	+
Operational Impacts to Rail Traffic	+		-	-	-	-
Operational Impacts to Marine Traffic	+	+	+			
Reliability	-			+	+	+
Fracture Critical Members	-	-	-	-	-	+
Seismic Performance	-	-	-	-	+	+
Elimination of Pin Connections	-	-	+	+	+	+
Environmental Impacts	+					
Corrects High Tower Deficiencies	-	+	+	+	+	+
Historic Impacts	+	+			-	-
Maintenance	-	-	-		+	+
Initial Cost	+				-	-
Annualized Cost	N/A	+			-	

Legend:
+ = Comparative Advantage
- = Comparative Disadvantage
[blank] = Negligible Comparative Advantage/Disadvantage

Several important points that should be noted when considering the advantages and disadvantages of the alternatives:

- Only Alternatives IIIa, IIIb, IVa, and IVb address the previously noted concerns with the pin and eyebar connections of Spans 5, 6 and 7. Fracture critical members will remain for all alternatives: tension members of the trusses for Alternatives I thru IVa, and the bottom flanges of the two girder deck system of Alternative IVb. Designs will be developed to address fatigue associated with fracture critical members;
- Alternatives IIIb, IVa, and IVb will facilitate higher allowable speeds across the bridge due to the entirely new superstructure. If coupled with other improvements on the New Haven Line, this will result in a time savings for passengers and be consistent with Amtrak's extensive program to upgrade the corridor. User cost benefits associated with these improvements are not included in this analysis, as other improvements outside the project limits are necessary for the higher speeds on the New Haven Line to be realized;
- The annualized long term maintenance costs for Alternatives II and IIIa are significantly higher than those of Alternatives IIIb, IVa, and IVb, as existing superstructure components are retained;
- Alternatives I and II offer considerably less construction impacts to rail operations due to the shorter construction duration. In particular, Alternative I repairs will have impacts to rail operations of less than one year due to the nature of the repairs that will be performed. Alternatives IIIa through IVb will significantly impact operations for up to four years while the work on the bridge is being completed;
- Alternatives IVa and IVb will result in the largest amount of long term environmental impacts due to the installation of the new piers in the river. Alternatives IIIa and IIIb also present long term environmental impacts due to the encasement of the existing piers in concrete. Alternatives II through IIIa will also have possible short term environmental impacts due to the cleaning and painting of the existing structure;
- Because the bridge consists of two largely independent structures with work generally isolated between two stages, it is possible for the entire project to be staged to match available funding. While this is not recommended due to increased mobilization and other procurement costs, Stage 1 (Tracks 1 and 3) work could be bid as one project, with the Stage 2 (Tracks 2 and 4) bid at a later date. Further investigation regarding the funding sequence would be required to determine possible conflicts, overlap, and timing issues between the stages. For example when considering Alternatives IVa and IVb, the waterway area will be restricted to a minimum after the cofferdam and piers of Stage 1 are installed before the existing piers are removed. The hydraulic adequacy of this condition will need to be investigated based on the time period anticipated before the existing piers are removed and the cofferdams removed; and
- Alternative IVa and IVb will allow for a much larger navigation channel due to the increased length of the movable span of the Devon Bridge and coordination with future pier locations of the Moses Wheeler Bridge.

Specific advantages and disadvantages of each alternative are listed in the following page.

Alternative Advantages and Disadvantages

Alternative	Advantages	Disadvantages
I – Short Term Repair	• Lowest initial construction cost • Least impacts to rail operations during construction • Least impacts to marine operations during construction • Least impacts to adjacent station operations during construction • Shortest construction duration • Lowest environmental impacts • Bridge retains historical value • Lowest construction cost	• Additional work required in 5-7 years upon completion and thereafter • Labor intensive construction methods required • Continued on-going maintenance and operation cost associated with bridge • No seismic retrofit to address vulnerability to seismic events • Does not address High Tower inadequacies • Piers remain susceptible to scour • Retains fracture critical pin and eyebar connections at Spans 5, 6 & 7 • Speed of rail traffic will remain slow, with the lowest reliability of bridge performance
II – Rehabilitation	• Low initial cost • Low annualized cost • Second least impacts to marine operations during construction • Bridge retains historical value	• Additional rehabilitation required in 25 years • Long term disruption to rail operations • Labor intensive construction methods required • Continued on-going maintenance and operation cost associated with bridge • Environmental impacts due to painting • No seismic retrofit to address vulnerability to seismic events • Retains fracture critical pin and eyebar connections at Spans 5, 6 & 7 • Requires installation of scour monitoring device or scour countermeasures • Speed of rail traffic will remain slow
IIIa – Partial Superstructure Replacement	• Less impacts to marine operations during construction • Construction duration on-site can be minimized presuming new trusses are constructed off site and floated into place • Eliminates fracture critical pin and and eyebar members • Existing bridge appearance and configuration can be replicated to some extent	• Additional rehabilitation of Spans 1-4 required in 25 years • Extensive retrofit to substructures required for seismic compliance • Continued on-going maintenance and operation cost associated with bridge • Environmental impacts due to painting • Historic impacts due to partial replacement of existing superstructure and modifications to substructure • Modifications to existing piers will increase projected waterway area, possibly adversely affecting flood elevations • Requires installation of scour monitoring device or scour countermeasures
IIIb – Full Superstructure Replacement	• Increased reliability of operations due to new movable span • Construction duration can be minimized if new trusses are constructed off site • Existing bridge appearance and configuration can be replicated to some extent • Eliminates fracture critical pin and and eyebar members	• Extensive retrofit to substructures required for seismic compliance • Historic impacts due to replacement of existing superstructure and modifications to substructure • Modifications to existing piers will increase projected waterway area, possibly adversely affecting flood elevations • Requires installation of scour monitoring device or scour countermeasures
IVa – Complete Structure Replacement with Trusses	• New structure with 75+ year life span • Increased reliability of operations due to new movable span • Minimal future maintenance costs • Seismically adequate • Foundation designed for vessel collision • Reduced overall pier width will increase waterway opening and improve river hydraulics • Somewhat replicates appearance of existing truss structure • Can accommodate high speed rail service	• Highest initial cost • Highest annualized cost • Environmental impacts associated with new piers • Historic impacts due to replacement of existing structure • Retains fracture critical and non-redundant truss system for entire bridge
IVb – Complete Structure Replacement with Deck Girders	• New structure with 75+ year life span • Lesser initial cost of the two complete structure replacement alternatives • Lesser annualized cost of the two complete structure replacement alternatives • Increased reliability of operations due to new movable span • Minimal future maintenance costs • Eliminates fracture critical pin and and eyebar members • Seismically adequate • Foundation designed for vessel collision • Can accommodate high speed rail service	• Environmental impacts associated with new piers • Historic impacts due to complete change of structure type and layout • Additional piers in waterway may have adverse hydraulic effects • Visual impact of new structure may be problematic

H. Recommendations

Based on a review the advantages and disadvantages of the six alternatives, Alterantive IVb – Full Replacement with Deck Girders appears to be the most advantageous rehabilitation option. While not the least expensive option in terms of both initial cost as well as annualized cost, this option will provide an entirely new and reliable structure that will be designed entirely in accordance with current codes and standards and meet the future needs of the rail corridor. The cost annualized cost differential between the next most reasonable alternative (Alternative IIIb) is approximately 13%, and the initial costs of each are comparable as well.

The work proposed with Alternative IVb will:

- Be composed of an entirely new structure with a 75+ year life span;
- Address deficiencies of pin and eyebar connections;
- Facilitate more conventional superstructure erection techniques (ie crane picks);
- Be seismically adequate;
- Be designed for scour;
- Be designed for vessel impact;
- Improve reliability of the bridge structure and movable span;
- Facilitate possible future higher speeds through the corridor;
- Provide for less future maintenance;
- Allow for increased navigation channel width;
- Correct deficiencies in the High Towers; and
- Be consistent with current efforts to upgrade the Northeast Corridor to a high speed rail facility.

II. Introduction

A.) Purpose of Report

The purpose of this report is to investigate the feasibility, constructability and costs associated with various alternatives to repair, rehabilitate, or replace the Metro-North Bridge over the Housatonic River in Stratford/Milford, Connecticut (Devon Bridge) (CTDOT Bridge No. 8080R, M.P. 60.42).

Stantec completed an in-depth inspection of the structure between May 4, 2009 and June 25, 2009, with additional days of inspection and verification of findings through September 2009. Subsequent to the inspection, Stantec analyzed the existing structure, developed alternatives for the repair, rehabilitation, and replacement of the bridge, and prepared life cycle cost analyses associated with each of the alternatives. This report presents a summary of the findings relative to the inspection and load rating analysis, summaries of repair, rehabilitation and replacement alternatives, and descriptions of repair and rehabilitation procedures and preliminary bridge type studies. All alternatives are evaluated for their cost, constructability, life cycle, and impact to railroad operations.

B.) Report Format

This report is divided into the following sections:

Abstract – This section provides a concise summary of the full report.

Section I – Executive Summary – This section provides an overview summary of the full report.

Section II – Introduction – This section outlines the purpose of the report, the description of the proposed project, an overview of the layout and configuration of the existing bridge, and a brief overview of the history of the bridge.

Section III – Methodology and Procedures – This section describes the overall methodology used to develop the findings of the report, as well as the parameters and assumptions imposed when developing the findings.

Section IV – Inspection Findings and Load Ratings – This section summarizes the findings of the in-depth inspection of the bridge and high towers and provides a summary of results from the load rating for the bridge and analysis of the high towers.

Section V – Railroad Operations – This section describes the current service, configuration, and physical geometry of the rail line in the vicinity of the project area. This section also outlines several alternatives for handling rail traffic during construction.

Section VI – Marine Operations and Navigational Requirements – This section describes the requirements for marine traffic in the Housatonic River.

Section VII – Scour Assessment and Hydraulics – This section describes an overview of the existing hydraulic conditions and an assessment of scour potential of the existing structure.

Section VIII – Soils and Foundation Assessment – This section describes subsurface conditions in the vicinity of the bridge, provides a summary of the existing substructure system, and outlines proposed substructures for each of the alternatives developed.

Section IX – Environmental Concerns – This section provides a summary of environmental issues and anticipated permitting requirements.

Section X – Seismic Assessment – This section outlines the results of a seismic assessment of the existing structure.

Section XI – Design Alternatives – This section presents a narrative description of each of the five rehabilitation/replacement alternatives as well as graphical representations of each. Sequencing, constructability, rail and marine operational impacts, and other significant impacts are discussed for each alternative.

Section XII – Alternatives Cost Analysis – This section provides preliminary cost estimates for each of the alternatives as well as a life-cycle cost analysis comparing the alternatives.

Section XIII – Recommendations – The final section discusses the results of all investigations and findings, and provides a recommendation as to the most appropriate repair, rehabilitation, or replacement alterative for the project.

C.) Description of Project

The Metro–North Railroad (MNRR) New Haven Line provides commuter and freight rail service between New Haven, CT and New York City, NY over a three and four-track electrified railroad system. Within Connecticut, the rail line and right-of-way is owned by the Connecticut Department of Transportation (CTDOT). The railroad line crosses the Housatonic River between Stratford and Milford, Connecticut on four tracks. The bridge over the Housatonic River, known as Devon Bridge, is a seven-span, open floor, steel truss structure. The fourth span from the west consists of a bascule lift movable span. The Bridge is owned by CTDOT and operated and maintained by MNRR.

The project limits for the feasibility study of the Devon Bridge extend from approximately milepost (MP) 56.50 eastbound through the bridge to approximately MP 61.00. The project includes the rail bridge proper, its approaches, navigation channel appurtenances under bridge, and three High Tower structures carrying signal, communications, and power facilities.

Stantec Consulting Services Inc. (Stantec) has conducted an Engineering, Feasibility and Economic Analysis of the Metro-North Railroad Bridge over the Housatonic River (Devon Bridge), Bridge No. 08080R, M.P. 60.42.

The bridge is an integral component of the Metro-North New Haven Commuter Line and Amtrak Northeast Corridor Line, carrying an estimated 6,300 passengers daily across the Housatonic River. Constructed in 1905, the existing bridge underwent a major rehabilitation in 1990 and is subject to on-going maintenance efforts. Based on its age, condition, serviceability, and current maintenance requirements, CTDOT has begun to develop a program to ensure the existing crossing of the Housatonic River is maintained for rail traffic in the future.

In support of this effort, Stantec completed an in-depth inspection, analyzed the existing structure, developed alternatives for the repair, rehabilitation, and replacement of the bridge, and prepared life cycle cost analyses associated with each of the alternatives. The intent of this report is to summarize the findings of these efforts, and to provide CTDOT with a comprehensive report to allow for management decisions regarding the future allocation of funds relative to the repairs and long term options associated with the bridge.

D.) Bridge Description

The Devon Bridge carries the Metro-North New Haven Commuter Line and the Amtrak Northeast Corridor Line over the Housatonic River at the Stratford and Milford town line. Located at milepost 60.42 along New Haven's Commuter line, the bridge is immediately west of the Waterbury Branch turnout. The bridge is comprised of parallel twin structures, with each structure carrying two active tracks. The northerly structure carries Tracks 1 and 3, while the southerly structure supports Tracks 2 and 4. A single leaf bascule spans over the navigable channel of the Housatonic River.

1. <u>Superstructure</u>

Structurally, the bridge consists of a seven (7) span, open deck structure with an overall length of 1,067 feet. Spans are numbered 1 through 7 consecutively from west to east, and have overall span lengths of 146.33 feet, 110.42 feet, 34.37 feet, 110.00 feet, 222.58 feet, 222.42 feet and 220.58 feet, respectively. With the exception of Spans 2 and 3, the spans consist of parallel-twin trusses. Each pair of trusses supports two active tracks. The northernmost trusses support Track Nos. 1 and 3, and the southernmost trusses support Track Nos. 2 and 4. The railway is on an open floor with continuous welded rails on timber ties. The tracks are supported by built-up riveted stringers in Span Nos. 1, 4, 5, 6 and 7 and by built-up riveted girders in Span Nos. 2 and 3. The tracks are spaced at 13 feet center to center. Refer to Figures G-1 and G-2.

Photo II-1: Spans 5, 6, and 7

Photo II-2: Spans 1, 2, 3, and 4

Span 1 consists of gusset-plated Warren trusses with a floorbeam and stringer flooring system. Span 2 consists of riveted deck girders. Span 3 consists of deck girders supporting end and mid-span floorbeams with stringers. The girders also support the rolling lift during bridge opening events. Span 4 consists of a "Scherzer Rolling Lift" through truss with floorbeams and stringers. Truss members are connected with gusset plates and rivets. Spans 5, 6 and 7 consist of pin-connected Baltimore trusses and a floorbeam and stringer flooring system. Built-up compression members and multiple flat plate eyebar tension members form the truss.

Photo II-3: Span 4

2. Substructure

The bridge is supported by two gravity type masonry abutments founded on spread footings, with six intermediate piers constructed of masonry with deep concrete foundations. The abutments are labeled Abutments 1 and 2, from west to east. The piers are labeled Piers 1 through 6, again from west to east. A timber pile supported fender system protects Piers 3 and 4, which are adjacent to the navigation channel below movable Span 4.

Photo II-4: Typical Masonry Pier

3. Track

The track system consists continuous of welded rail fastened to wooden ties with Pandrol 'e' Clip style fasteners, tie plates, and spikes. The ties are blocked and strapped together throughout, and bolted directly to the top flange of the stringers or girders. Square cut miter rail joints with outside rider rails are present at the movable span.

4. Mechanical

Span 4 opens for marine traffic by rolling back away from the channel westward on tracks mounted to the main girders and pier structure of Span 3. The racks on the main girders on Span 4 are curved with pinions at their center of rotation, and tracks on the pier structure are horizontal.

The operating machinery for each bascule span is located on the west end above the catenary system. The span drive machinery for each leaf consists of twin electric motors coupled to the input shafts of an enclosed differential speed reducer. During operation, one of the two motors is selected to operate a leaf. Each leaf has two motor brakes and two machinery brakes. All brakes are functional at all times. Each enclosed speed reducer drives three sets of open gearing on each side of the span. The final set of open gearing consists of a pinion riding over a horizontally mounted straight rack.

The toe of each leaf is supported by a live load shoe on each side with a vertical projection used to center the leaf during seating. The span lock machinery for each leaf consists of a single electrical gearmotor that drives an open gearset. The open gearset drives a series of cranks and linkages. The final linkage is a hook that extends under a pin on each side of the span to lock the span in the seated position. Each span has a single air buffer intended to cushion the span in the event of a harsh seating operation.

5. Electrical

The bridge is composed of two parallel rolling leafs with a control system capable of independent operation of each leaf. The leafs are sequentially opened for safety against collision of marine traffic. In addition, the Bascule clearance navigation lights are tied to the two leafs such that they will transfer from red to green only when both leafs are fully raised. One leaf can be opened while train traffic is conducted on the other.

Power is supplied from a transformer west of the bridge. The bridge is provided with an automatic transfer switch but no alternate feeds or standby generator. Power is brought to the electrical room at the operator's level and distributed to the various systems including:

- Automatic transfer switch
- Power distribution panel
- Lighting distribution panel
- Four span motor drives
- Two motor control centers
- Operator's control console
- Programmable controller systems

The bridge control system is interlocked with the signal system to assure that the bridge can only be opened if the derails are set for both tracks of that leaf. The interlock also controls and monitors contactors in the motor control centers such that 480 volt motor power is not available to run motors unless the signal system unlocks the control system. Each bridge is interlocked separately such that one bridge can be opened while the other is available for train traffic.

Each leaf control and drive system and electrical control system is composed of the following:

- Two wound rotor span drive motors with secondary resistor
- Two thrustor type motor brakes and two thrustor type machinery brakes
- Two span motor tachometers and speed switch pair for each span drive motor
- Disconnect switches for every motor including span drive motors and brake motors
- One rotary cam type span limit switch
- Two plunger type span seated switches
- One motorized lock bar actuator with brake motor and disconnect for motor and brake
- One rotary cam type lock bar limit switch

6. Operator's House

The Operator's House is located between Piers 2 and 3 on the south side of the bridge. Access is provided by walking across bridge along Track 4, and then climbing a series of steel ladders to the Operator's House level. The Operator's House houses the bridge controls as well as electrical components associated with the movable span. The Operator's House was rebuilt as part of the 1990's work on the bridge.

The structure is built using conventional wood framing supported by steel sub-framing. A steel deck wraps around the west, south, and east sides of the house.

Support framing for the Operator's house consists of three levels including the house floor framing, and upper platform and a lower platform all supported by truss towers on four sides. All three levels include a series of rolled W and C shapes with either a cast in place concrete deck (house level) or wood plank flooring. The vertical support framing consists of riveted built-up column members at the four corners braced by angle cross frames and lattice girders.

7. High Towers

There are three high towers associated with the Devon Bridge. Two towers are located immediately west and east of the bridge and are integral with Catenary Towers 862 and 863, with the third tower located between Spans 4 and 5.

Photo II-5: Bridge Span and High Tower Numbering (looking north)

The towers carry Metro-North Railroad high voltage feeder lines, catenary jumper lines, and catenary wires along with United Illuminating high voltage power lines over the Housatonic River at the Stratford/Milford town line. Note that the middle tower does not carry catenary wires and does not have an official catenary number designation. For the purposes of this report, it is referred to as Catenary 862A. Catenary 862 is located west of the bridge in Milford, Catenary 862A is located on the bridge on pier 3 just east of the bascule span, and Catenary 863 is located to the east in Stratford. The structures were built in 1912. All of the towers rise approximately 198.50 feet above ground level and are connected at three levels by truss bridges. The lower, middle, and upper bridges are 24.50 feet, 99.67 feet, and 170.50 feet above ground level. The tower legs are comprised of steel angles with cover plates braced together with smaller angles and channel sections. The towers are supported on reinforced concrete foundations. The bottom 1½ panels of Catenary 862A are encased in concrete.

E.) Historical Information

Historical information contained herein was obtained from the May 2007 report by Fitzgerald & Halliday, Inc.

Since 1849, when the railroad was completed between New York and New Haven, there has been some form of railroad-bridge over the Housatonic River at this former ferry crossing. Originally a single track, the line was double tracked in 1854. Up until the current bridge was built (c.1905), the bridge over the Housatonic (known as the John McMahon Bridge) consisted of six fixed deck truss double-track spans and one double-track swing span.

By the 1890s, the trackage on the main line was insufficient to meet demand, causing major delays. In 1896-1897, the main line from Woodlawn Junction, in New York, to New Haven was widened to four tracks. However, due to cost and logistics, it was nearly a decade before three four-tracked bridges over major crossings—the Mianus in Cos Cob; the Saugatuck in Westport; and the Connecticut at Warehouse Point– were completed.

Not only were the bridges not wide enough, they could not support full train loadings. When the New York, New Haven & Hartford Railroad Company's (NYNH&H RR) bridge specifications provided for a loading of two engines weighing 219,000 lbs with tenders weighing 112,000 lbs (known as an E-50 rating), the bridges had weight limits of 157,000 pounds (lbs) for engines and 100,000 lb cars were "not allowed to run over them loaded to their full capacity."

By the time the Devon Bridge was in the planning stage, the Scherzer rolling lift bridge type was in widespread use and was the primary type of movable bridge used by the NYNH&H RR. By 1905, more than 40 bridges of this type were in use throughout the country. Invented and patented in 1883 by William Scherzer (1858-1893) of Chicago, the relative simplicity of the lift mechanism, and the minimal power required to lift the bridge, were not the only features that appealed to the railroads. Since the lift span rolled away from the navigation channel as it was raised, it did not have rise as far as other types of lift bridges. This reduced the arc of swing and the amount of time the bridge had to remain open. The bridge could also be expanded by adding additional leaves, permitting continued operation during expansion, where swing bridge enlargement would require the construction of a temporary bypass and the complete scrapping of the existing span and its mechanical system.

Bridge drawings available at the CTDOT archives in Newington, Connecticut indicate that the bridge engineers of NYNH&H Railroad designed the fixed spans for the bridge in April of 1904. The drawings were signed for the railroad by Colin M. Ingersoll (1858-1948), Chief Engineer, and William H. Moore, Bridge Engineer. Ingersoll was a graduate of Yale (1880), joining the NYNH&H RR as an assistant engineer in 1881. In 1900, he was made Chief Engineer, and left the railroad in 1906 to become Chief Bridge Engineer for the New York City Department of Bridges. William H. Moore was born in Limerick Ireland in 1860 and received his engineering education at Queen's College, Cork and the Royal University in Dublin, receiving a first class honors degree from at Queen's College and a Master of Engineering from the Royal University. He arrived in the United States in 1885, and in 1886 was employed by the NYNH&H RR as a draftsman. In 1889, he was named Engineer of Bridges for the NYNH&H RR. He remained with the railroad until 1918.

The Scherzer Rolling Lift Bridge Company, the successor firm of William Scherzer, signed a contract in 1904 and completed the design of the lift span and its mechanisms by the end of the summer of 1904. The company was based in Chicago and was known for producing bridges that were inexpensive because they were "the extreme of simplicity". The bridge was constructed by the American Bridge Company, which was founded in April 1900 as part of a J. P. Morgan-led consolidation of some of the country's largest steel manufacturers and builders. The American Bridge Co., based near Pittsburgh, Pennsylvania, was responsible for some of the country's most notable spans including the San Francisco Bay Bridge. They pioneered steel as a construction material which led to its use in a variety of applications including buildings, bridges and vessels. American Bridge continues to operate on a world-wide basis today.

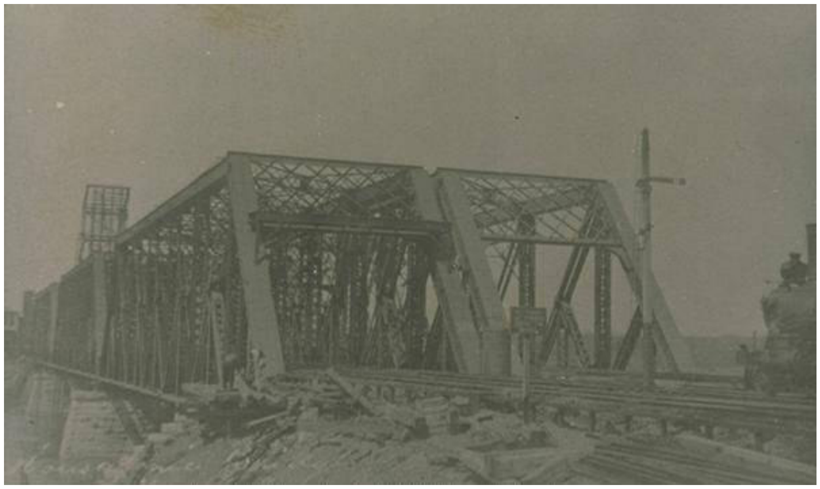


Photo II-6: Construction of Devon Bridge, c. 1905

Photo II-7: South Elevation of Devon Bridge, date unknown

Over the last 100 years, the bridge has undergone numerous repairs and rehabilitations. Steel repairs have been performed on the movable span and support steel. The rocker bearings were replaced with sliding plate bearings, deteriorated rivets were replaced with high strength bolts, and miscellaneous repairs to the segmental girders, rack girders and a rack adjustment were performed. Existing stone masonry piers and abutments have been re-pointed.

In 1963, the State of Connecticut created the Connecticut Transportation Authority (CTA) to study passenger service and preservation strategies to keep the New Haven line operational. In 1965, the CTA entered into an agreement with the NYNH&H RR, the Metropolitan Transportation Authority (MTA) and the U.S. Department of Housing and Urban Development (HUD) to subsidize the passenger service for an 18-month pilot program. Under this program, passenger service was privately operated with public funding until the end of 1966, and was funded by the CTA and the MTA continued until the end of 1968. Passenger service continued to be operated by Penn Central until 1970, when the railroad filed for bankruptcy protection.

Consequently, CTA, now the Connecticut Department of Transportation (CTDOT, renamed in 1969), and the MTA, in conjunction with the bankruptcy trustees of the Penn Central, immediately stepped in and took control of passenger service operations. The agreement stated that CTDOT had the option to lease all Connecticut rights-of-way leading to New Haven, New Canaan, Danbury and Waterbury. This agreement would remain in effect for 60 years and CTDOT and the MTA would equally share costs associated with operating, maintaining, and improving the right-of-way. Over the next six years, improvements of the passenger service were implemented, including rehabilitation of the depleted fleet and infrastructure. The Railroad Revitalization and Regulatory Reform Act of 1976 formed the Consolidated Rail Corporation (CONRAIL) to take over operations of defunct and bankrupt railroads in the Northeast. CONRAIL took over assets of the former Penn Central, a partner in the 1970 agreement to operate passenger service on the New Haven Line. In 1981, the Northeast Rail Services Act relieved CONRAIL from operating passenger service on the New Haven Line. CTDOT and the MTA assumed passenger operation of the line themselves. The MTA created the Metro North Commuter Railroad (MNCR) to operate the New Haven Line for CTDOT – MTA partnership. MNCR took over operations on January 1, 1983. In 1985, CTDOT amended their agreement with the MTA by changing the cost-sharing ratio of the New Haven Line. CTDOT now provides 56 percent of the New Haven Line operating deficit and 53 percent of the operating deficit associated with New Haven Line trains in Grand Central Terminal. CTDOT also assumed responsibility for all capital costs associated with the New Haven Line. In addition, CTDOT exercised its right to purchase the New Haven Line right-of-way in Connecticut.

The bridge has undergone numerous repairs and rehabilitations over the past century. It was significantly rehabilitated in 1990. Steel repairs were mostly performed on the movable span and support steel. The rocker bearings were replaced with sliding plate bearings, deteriorated rivets were replaced with high strength bolts, and miscellaneous repairs to the segmental girders, rack girders and a rack adjustment were performed. Existing stone masonry piers and abutments were repointed and strengthened with steel bars.

The Devon Railroad Bridge was placed on the National Register of Historic Places in 1987. This bridge, along with seven other bridges located on the Northeast Corridor in Connecticut, was identified in an aerial reconnaissance survey of historic and archeological resources undertaken in 1987 as part of the Northeast Corridor Improvement Project. In 1990, it was rehabilitated as part of the same project. The seven other railroad bridges that were listed were the Mianus River (Cos Cob), Norwalk River (South Norwalk), Pequonnock River (Bridgeport), Saga Bridge (Westport), Connecticut River (Old Saybrook), Niantic River (East Lyme), and the Thames River (Groton).

F.) Utilities

There are existing utilities on and in the vicinity of the bridge, consisting primarily of electrical and communications services. There are also various utilities in the area proximate to the railroad which are outside the limits of the bridge.

The majority of the utilities associated with the bridge are located on the High Towers. These include:

- Ground Cables (United Illuminating)
- High Voltage Power (United Illuminating)
- Ground Cables (MNR)
- High Voltage Power (MNR)
- Feeder Cable (MNR)
- Signal Wires (MNR)
- Track Messengers Wires (MNR)

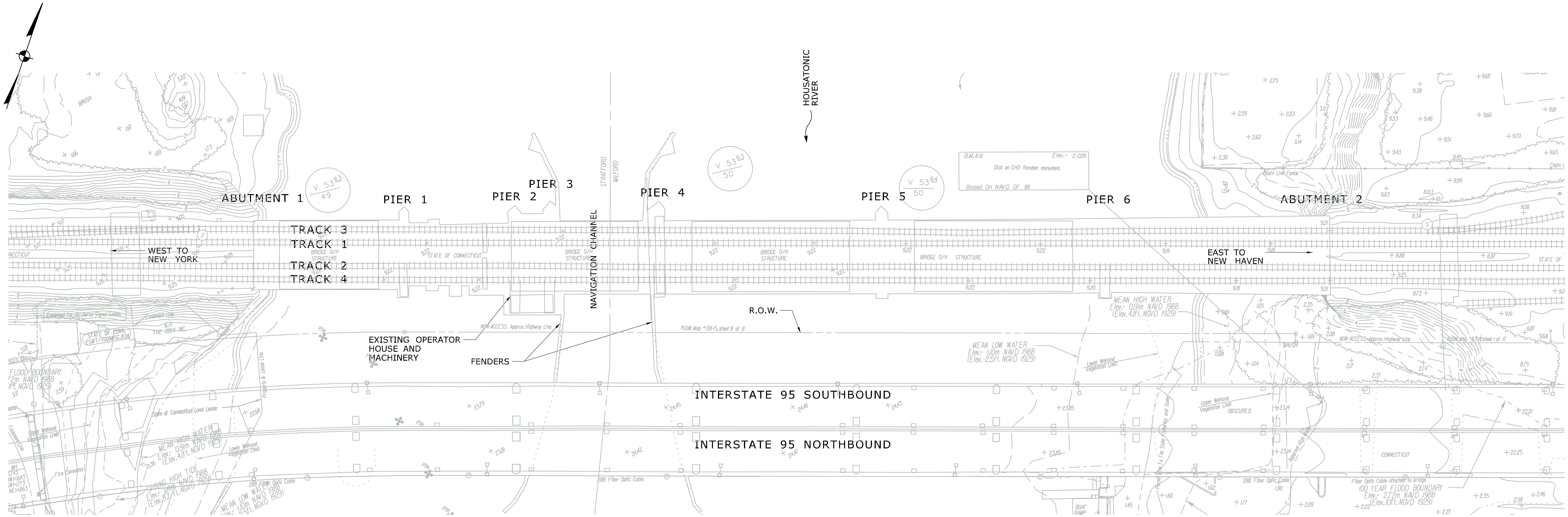
- Control Wires (MNR)

On or attached to the bridge itself are:

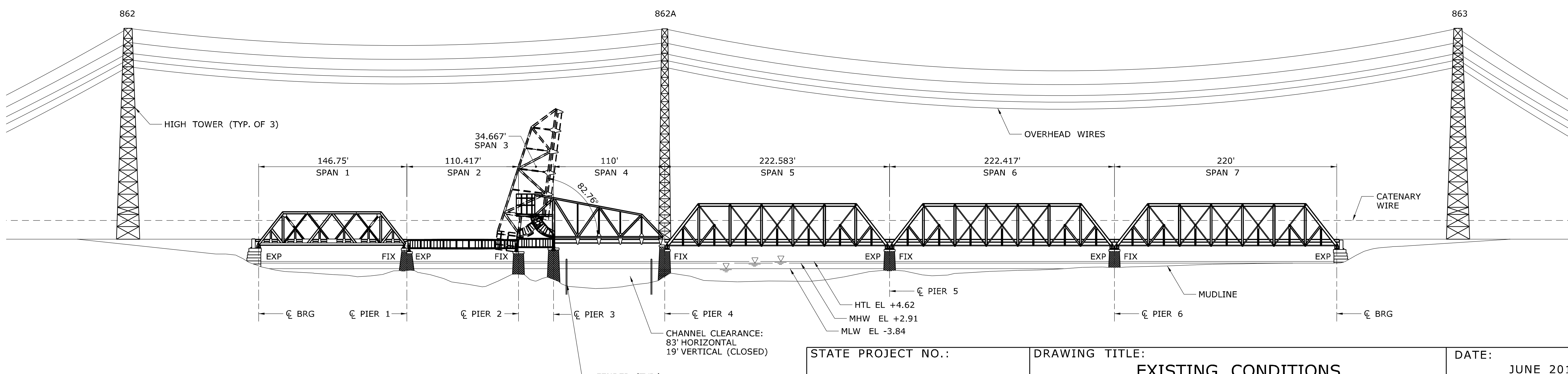
- Signal Cables (MNR)
- Power Cables for luminaries
- Power Cables for Operator's House
- Telephone for Operator's House

MNR personnel also indicated that there is a UI power cable across the navigation channel with approximately 70 feet of clearance.

About one quarter mile to the east of the bridge, 345kV Transmission Duct Bank (Northeast Utilities) crosses under the tracks, and turns west and runs parallel to the bridge on the south side of I-95.

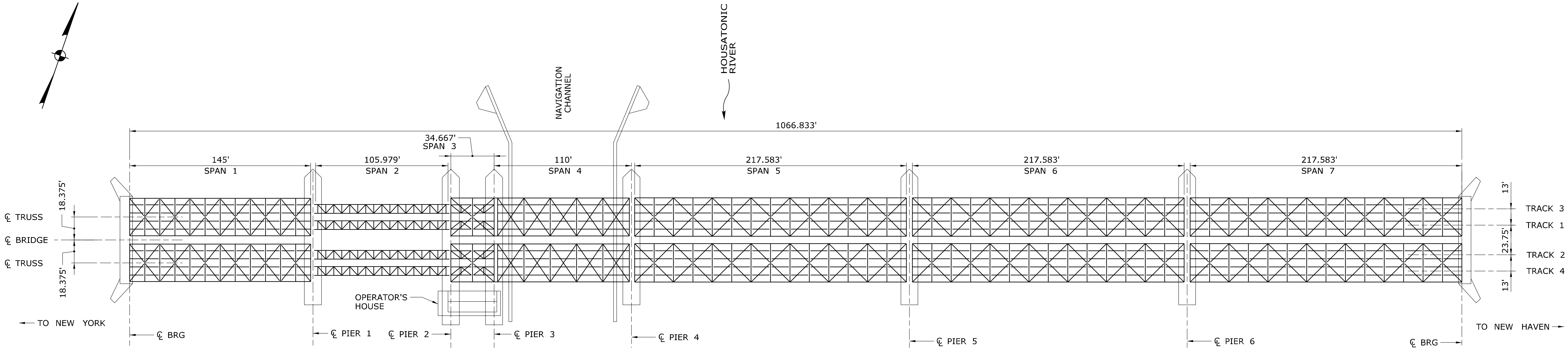


PLAN

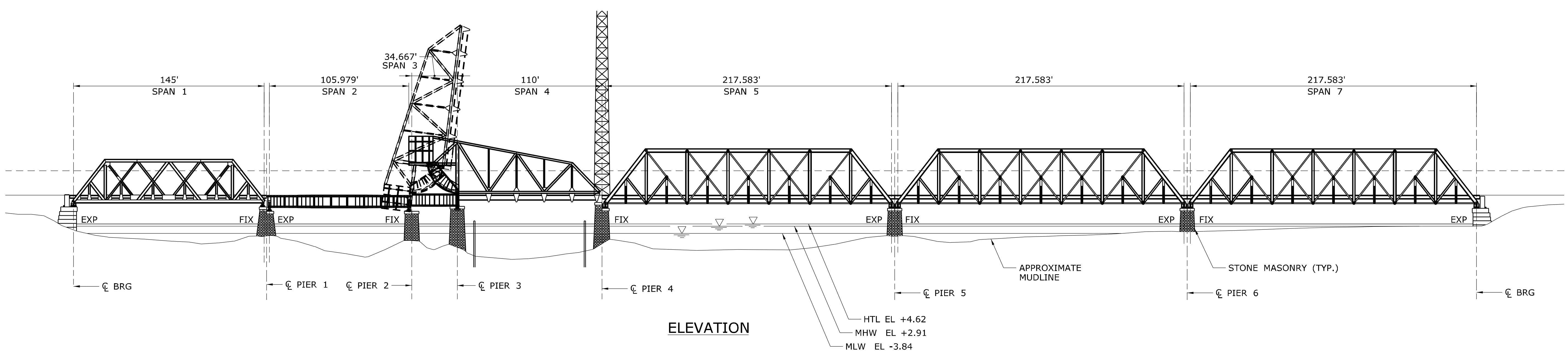


ELEVATION

STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	EXISTING CONDITIONS	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE:
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	1" = 100'
		FIGURE: G-1

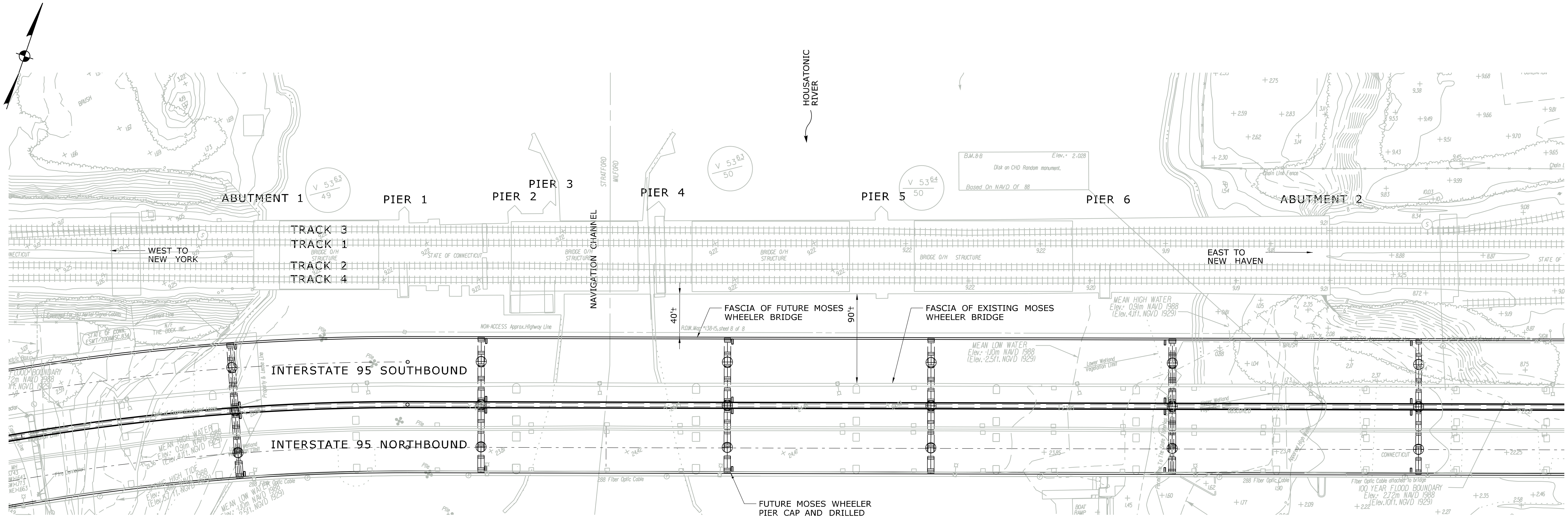


FRAMING PLAN

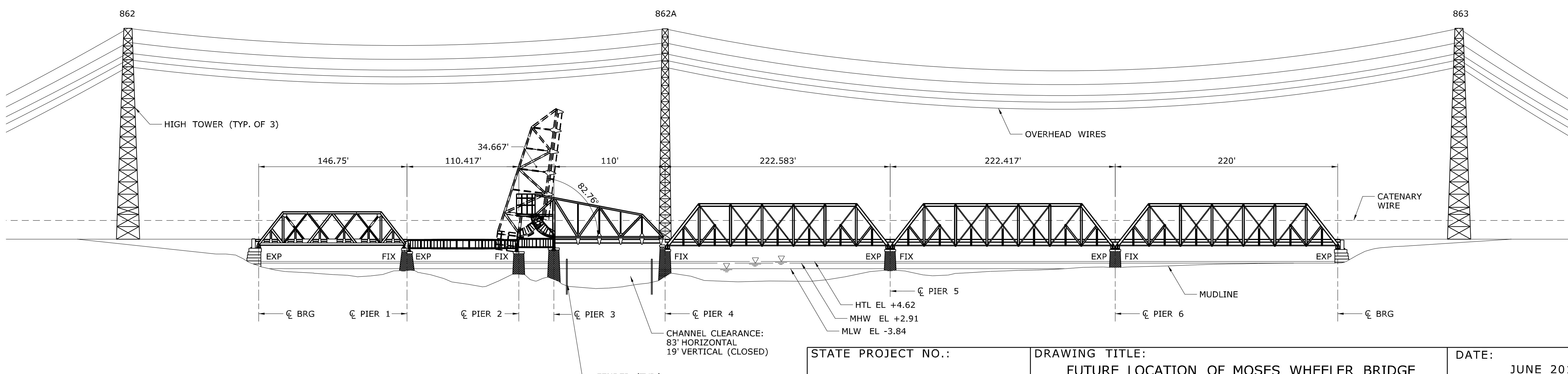


ELEVATION

STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	EXISTING CONDITIONS	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE:
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	1" = 80'
		FIGURE: G-2

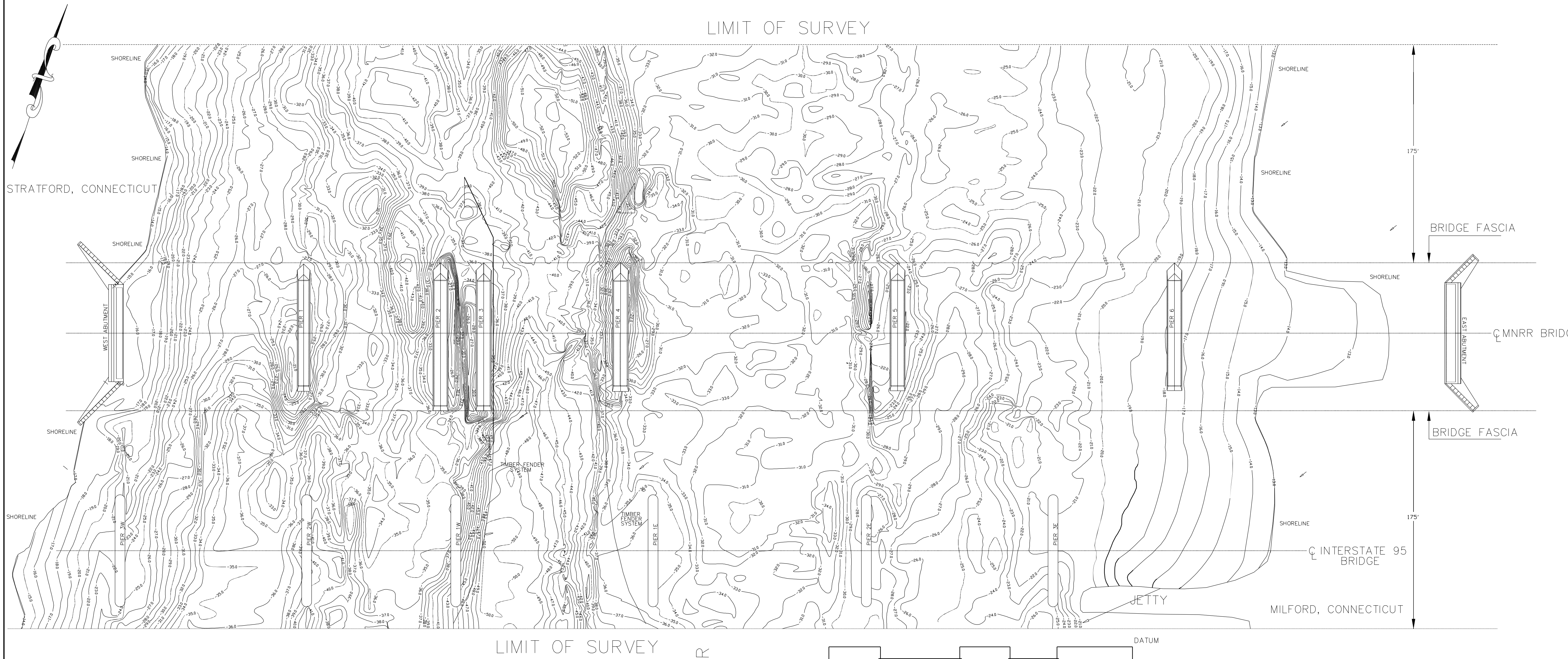


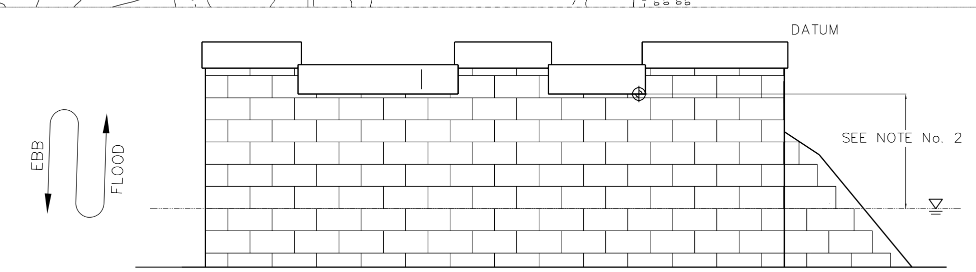
PLAN



ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: FUTURE LOCATION OF MOSES WHEELER BRIDGE	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 100'
		FIGURE: G-3





PIER No. 1 EAST ELEVATION

⊕ DATUM ELEV. 0.0 TAKEN FROM BOTTOM OF LOWER CAP STONE AT THE NORTHEAST CORNER ON EAST FACE OF PIER NO. 1.

∇ WATER SURFACE ELEVATION

NOTES:

1. THE INFORMATION PRESENTED ON THIS DRAWING REPRESENTS THE RESULTS OF A FATHOMETER SURVEY PERFORMED BY A. DICESARE ASSOCIATES P.C. DURING THE PERIOD OF MAY 21 TO MAY 22, 2009. IT CAN ONLY BE CONSIDERED AS INDICATING THE CONDITIONS EXISTING AT THE TIME OF THE SURVEY.
2. THE HOUSATONIC RIVER IS TIDALLY INFLUENCED. THE WATER SURFACE WAS CONTINUALLY MONITORED DURING THE SURVEY AND RANGED FROM −16.8' TO −13.1' BELOW BENCHMARK AT TIME OF SURVEY. ALL ELEVATIONS ARE REFERENCED TO THE BENCHMARK ELEVATION.
3. THIS FATHOMETER SURVEY WAS PERFORMED TO EVALUATE SCOUR ACTIVITY IN THE VICINITY OF BRIDGE No. 08080R WHICH CROSSES THE HOUSATONIC RIVER. THE LIMITS OF THIS SURVEY EXTEND 175 FEET NORTH AND SOUTH OF THE BRIDGE FASCIAS.
4. THE CONTOUR INTERVAL ON THIS PLAN IS 1 FT.
5. BRIDGE STRUCTURES AND NORTH ORIENTATION ARE APPROXIMATE AND WERE PLOTTED FROM DRAWINGS PROVIDED BY CONNECTICUT D.O.T.

STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	FATHOMETRIC SURVEY	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE:
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	1" = 80'
		FIGURE: G-4

III. Methodology and Procedures

A.) Methodology

Stantec conducted a comprehensive evaluation of the existing bridge prior to commencing the alternatives analysis. This evaluation centered on determining the condition and structural capacity of the existing bridge and three associated electrical high towers. Additional investigations were also conducted prior to developing the alternatives analysis.

To assess the condition of the bridge, Stantec performed in-depth structural, mechanical and electrical inspections of the existing bridge and high tower components in accordance with the CTDOT Bridge Inspection Manual. Prior to commencing the field inspection, Stantec reviewed previous inspection, load rating, and various other reports to become familiar with anticipated field conditions, inspection procedures, and access requirements. Stantec, assisted by A. DiCesare Associates and Garg Consulting Services, then performed the field inspection primarily between May 4, 2009 and June 25, 2009, with additional days of inspection and verification of findings through September 2009. Inspection of the mechanical and electrical components required observation of the movable span in operation, which occurred on June 14, 2009. During the field inspection, inspectors also took measurements of critical structural components and associated losses due to deterioration. In addition, non-destructive testing of the bridge truss pins and an analysis of the existing paint system was also performed during this time period by specialty subconsultants.

Stantec conducted load rating analyses of each of the bridge spans based on field conditions noted during the in-depth inspection. The load rating analyses were performed in accordance with the 2007 Manual for Railway Engineering published by the American Railway Engineering and Maintenance-of-Way Association (AREMA). The structural model used to perform the structural analysis was based on the original design and shop drawings, subsequent rehabilitation drawings, and actual field measurements. Information from these sources was used to identify member section properties and material properties. All primary load carrying members within each of the seven spans were evaluated for both a baseline "As built" condition (assuming no member deterioration) as well as an "As inspected" condition (including member deterioration).

Members evaluated during the load rating included trusses, floorbeams, stringers, girders, and truss pins, using a standard Cooper E-80 live loading pattern as a basis for the analyses. Each member was evaluated for the aforementioned "As built" and "As inspected" conditions at Normal, Maximum, and Fatigue allowable stress levels for axial, moment and shear forces. In addition, Stantec also conducted a seismic analysis of the existing bridge to assess its vulnerability to seismic events.

Each of the three high towers were evaluated based on the National Electrical Safety Code (NESC) for three load cases including Combined Ice and Wind (NESC Load Case 250B), Extreme Wind (Case 250C), and Extreme Ice and with Concurrent Wind (250D). The structural model used during the structural analysis was based on plans on file for the high towers from the 1912 Stamford-New Haven Electrification project by the New York, New Haven & Hartford Railroad Company, except the bridges between tower legs which were constructed differently from those shown on the plans. Plans on file did not have specific information for the Devon Bridge, however the two side towers (862 and 863) were identical to others shown on the plans. No plans were available for 862A or for the bridges between the two tower legs at 862 and 863; elements comprising these structures were measured in the field or interpreted from photographs when inaccessible due to high voltage wires. The various tower members within each tower were evaluated for both a baseline "As built" condition (assuming no member deterioration) as well as an "As inspected" condition (including member deterioration).

Stantec also investigated various other aspects related to the bridge, as they contribute to its functionality and performance. These included analyzing the hydraulic and scour performance of the bridge in existing and future configurations, reviewing possible environmental and permitting requirements that would be necessary for the various rehabilitation alternatives, reviewing subsurface conditions, identifying potential historic impacts associated with modifying or replacing the existing bridge, evaluating marine navigational requirements, identifying potential utilities impacts, and finally, establishing both short- and long-term railroad operational requirements.

Based on the information collected and evaluated, Stantec then developed conceptual rehabilitation and replacement alternatives that partially or fully address deficiencies that were found. The alternatives were developed based on three service life horizons: 5-7 year, 25-year, and 75-year. The alternatives were segregated as such to allow for decisions regarding the future allocation of funds relative to the repairs and long term options associated with the Devon Bridge, including upgrading the corridor to high speed service.

B.) Design Codes

The basis of design for the preliminary repair, rehabilitation, and replacement concepts outlined in this report includes the following design codes:

- American Railway Engineering and Maintenance of Way Association (A.R.E.M.A), 2007 (the latest available report at available at the time of this report); and
- State of Connecticut Department of Transportation Bridge Design Manual, 2003.

The in-depth inspection of Devon Bridge summarized in this report was performed in accordance with the following standards and guidelines:

- State of Connecticut Department of Transportation Bridge Inspection Manual, 2001, Version 2.1;
- American Railway Engineering and Maintenance of Way Association (A.R.E.M.A.), 2007;
- Federal Railway Administration;
- United States Department of Transportation Federal Highway Administration (FHWA) Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, 1995; and
- American Association of State Highway and Transportation Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual, 2009. For general conformance only.

C.) Agencies, Regulations and Permits

In addition to the design codes referenced above, there are numerous local, state, and federal regulatory agencies that have jurisdiction over some or all of the various components associated with the rehabilitation and repair concepts considered in this report.

1. <u>United States Coast Guard</u>

The Rivers and Harbors Act of 1899, together with the General Bridge Act of 1946, require that the locations and plans of all bridges and causeways across navigable waterways be submitted to and approved by the Secretary of Transportation prior to construction. This approval was delegated to the United States Coast Guard (USCG) in 1967. As such, the USCG approves the location and plans of bridges and causeways and imposes any necessary conditions relating to the construction, maintenance, and operation of these bridges in the interest of public navigation. In addition, the Coast Guard is also required by law to ensure that environmental considerations are given careful attention and importance in each bridge permitting decision.

A USCG Bridge Permit will be required for any of the rehabilitation or replacement alternatives described in this report.

2. Connecticut Department of Transportation

The Connecticut Department of Tramsportation (CTDOT) is the owner of the bridge.

3. Connecticut Department of Environmental Protection

Devon Bridge is located in an estuary and construction along the site must comply with the Coastal Area Management Plan of the Connecticut Department of Environmental Protection (CTDEP). The State of Connecticut requires permits for structures, fill, and dredging in coastal areas. The CTDEP may waive the requirements of the state and may refer to the Corps of Engineers permits. Control of tidal wetlands, protection of endangered species, and regulations of sewerage discharges into tidal wetlands are included in a Coastal Management Plan.

4. Army Corps of Engineers

The United States Army Corps of Engineers administers regulations and procedures on discharge of dredged or fill material and transportation of dredged material for the purpose of dumping into the waters of the United States, based on the Clean Water Acts of 1972 and 1975 and the Marine Protection, Research, and Sanctuaries Act of 1972. The Corps of Engineers also has jurisdiction over any obstruction or alteration of navigable waters in the United States, based upon the Rivers and Harbors Act of 1899.

The chosen repair, rehabilitation, or replacement alternative may require obstruction of the navigable waterway, possible closure of the waterway for a length of time, and possible dredging of the channel after construction. A permit issued from the Corps of Engineers may be required prior to any alternative.

5. Connecticut State Historic Preservation Office

Although a permit is not requried from the Connecticut State Historic Preservation Office (SHPO), other regulatory agencies require consultation with this office prior to authorizing the proposed work. In accordance with Section 106 of the National Historic Preservation Act, and Section 4(f) of the DOT Act of 1966, consultation and coordination with SHPO will be required if a replacement or significant rehabliltation alternative were selected, requiring all reasonable mitigation measures to be completed prior to construction.

6. Housatonic River Estuary Commission

To be determined.

7. Other Agencies

The Connecticut Siting Council has jurisdiction over all construction and relocation of electric power transmission lines. In the case of the Devon Bridge, the electric power transmission lines carried on the existing catenary structures may have to be relocated onto temporary structures or transferred onto new structures during the construction period.

IV. Inspection Findings and Load Ratings

A.) Inspection Findings

1. **Structural**

In general, the bridge is in poor condition. The superstructure steel exhibits corrosion, pitting loss of section, general section loss, and impacted rust at various locations throughout. Driving the poor rating of the bridge are conditions of two components of the primary members: the stringers throughout the entire bridge and specifically those under Track 1, and pinned truss connections and associated eye bars of Spans 5, 6, and 7 which are extremely difficult to repair or replace. The stringers typically exhibit loss of section at the top flange in the form of pitting losses at the top plate and/or edge losses of the flange members (up to 100% losses at certain locations). Pitting type web losses were also noted in the stringers.

Photo IV-1: Typical Deterioration of Stringer Top Flange below Track 2

The bridge pins exhibit rotational movement at many locations throughout the bridge. This rotation has resulted in losses in the connecting eye bars due to the members rubbing against each other. The results of the ultrasonic testing examination of the pins did not reveal the presence of any crack-like indicators in any of the truss pins examined.

Photo IV-2: Typical Wear Between Members at Pins (plate cutting into

The balance of the structural steel (except for stingers and pinned connections) throughout the remainder of the bridge is generally in fair condition. The primary steel members generally exhibit corrosion, pitting, section loss, and impacted rust at various degree of severity. The connections between stringers and floorbeams generally exhibit moderate to heavy impacted rust and section loss at the bottom of the connection angles.

Photo IV-3: Section loss of bearing stiffener at G8, Span 2

Photo IV-4: Section at top of Floorbeam FB12, Span 1

The poor rating of the substructure components is due to exposure and deterioration of pier footings at Piers 2, 3, and 4. Although no undermining was noted, previous underwater inspection reports, along with the current inspection results, indicate active scour (aggregation and degradation) is occurring at all piers. Above the footings, the masonry pier stems as well as the abutment stems are in satisfactory condition, with minor loss of mortar and hairline cracks noted in the masonry.

Photo IV-5: Typical loss of mortar at pier joints

The paint system was found to have completely failed. As noted previously, the bridge is comprised of parallel twin structures. The northerly structure appears to have been repainted from the track level up, while portions below track level are unpainted or have completely failed paint. The southerly structure is completely unpainted. Lead was found in various paint samples taken from the bridge.

Photos IV-6 & IV-7: North (left truss) Truss painted above track level, South (right truss) truss unpainted (both photos looking East)

2. Track and Miter Rail

The rail and fasteners were noted to be in good condition. No fasteners were noted to be missing, and all tie plates had two spikes fastening it to the ties. The timber ties were generally in good condition. Isolated ties were noted at various locations with splitting and rot for up to the full width of the tie, with the majority of these noted in Spans 6 and 7.

Miter rails are located at the movable span rail joints. These joints are in fair condition. No missing bolts were noted. The rails exhibit no end batter at these joints. All headblocks exhibited moderate to excessive wear.

3. Mechanical

On May 10 and 11, 2009, June 14, 2009, and July 28, 2009 Stantec personnel performed a detailed inspection of the mechanical systems of the bridge. The conditions of both leaves were very similar. Therefore, the recorded conditions should be considered present in both leaves, unless otherwise noted.

The bridge machinery was found to be in generally good condition. The spans operate infrequently (approximately 95 times per year); the lack of operations has reduced wear but also appeared to have reduced the frequency of machinery lubrication.

Span Drive Machinery

The spans operate very infrequently for marine traffic (several times per month); this lack of operations has reduced wear but also appeared to have reduced the frequency of machinery lubrication.

Span operations during the inspection were only permitted on June 14, 2009. During operation of the north span the north and south intermediate gearsets were alternately driving the span. A possible cause of this may be unequal total backlash between the north and south side gearing. No unusual sounds were heard from the south leaf. Based on gear tooth contact, both spans appeared slightly span heavy during operation.

Tooth damage from tooth bottoming was present on the south end of the south rack and pinion of the north leaf ½”. The tips of the south pinion of the north leaf were ground down presumably to eliminate the bottoming. The extreme south ends were still bottoming. Lubrication of the racks and pinions was old, dried and inadequate. Portions of the teeth were rusting. Tooth alignment was inconsistent at different locations along the racks.

Photos IV-8 & IV-9: Wear and tooth damage to rack and pinion

External gear wear ranged from 1% to a maximum of 24%. The enclosed differential speed reducers were in good condition with little indication of wear.

Photo IV-10: Enclosed speed reducer

The main flange bearings, (B12 and B18), that support the rack and pinion shaft are all missing 2 of the 22 mounting bolts as previously reported. There was no evidence of bearing movement indicating that the installed 20 mounting bolts are adequate. Five of the eight brakes did not either fully set or fully release.

Span Lock Machinery

The span lock machinery operated smoothly and quietly. However, the system was heavily worn, corroded and no longer serves its original function of holding the span in the seated position.

The hooks and anchor pins were heavily worn and unpainted with corrosion. The span lock hooks and anchor pins all had excessive clearance and as such did not lock the span down in the seated position. It was reported by Metro-North personnel that if the hooks are adjusted to hold the spans down they get stuck when attempting to unlock the spans.

Span Support Machinery

The horizontal track plates and curved tread plates were in good condition.

Photo IV-11: Minor pintle wear noted on main tracks

The live load shoes were unpainted with heavy corrosion. Their anchor bolts were loose and heavily corroded. The north leaf south live load shoe pumped under train traffic and had a thin shim plate added over the pier shoe.

The centering devices consist of vertical bent plates attached to the live load shoes on the extreme north and south sides of each span. There was clearance at the north side of both spans and hard contact at the south side of both spans.

Photos IV-12 & IV-13: Span centering devices – clearance at North side, contact at South side

The air buffers were non-functional as previously reported. With the modern control system installed on the spans the air buffers are no longer necessary as the spans seat automatically.

4. Electrical

On May 10 and June 14, 2009, Stantec personnel performed a detailed inspection of the electrical systems of the bridge. The electrical system was found to be operational and in generally serviceable condition. The system is of previous-generation design, and at the time of the inspection, one operating system was out of service.

The following deficiencies were noted:

The electrical service is supplied from a transformer on the approach and brought to the control house via cables on messengers. There is no alternate power source available at the bridge.

The automatic transfer switch is in good condition, but its automatic operation is disabled as there is no alternate power source.

Photos IV-14 & IV-15: Incoming service panel (L), and Span motor drive crossover switch (R)

Photos IV-16 & IV-17: PLC (L), and control console (R)

The control console is located in the operator's room of the operator's house. The console equipment is fully functional. Position indications are accurate and show zero with the leafs seated. The voltmeter and ammeter are in good condition but the ammeter requires recalibration.

Each leaf is provided with redundant drive motors. The motor frames, mountings, brushes, brush holders and slip rings are in good condition. There was no evidence of water or grease entry into the interior of the motor. By the appearance of dry grease on the fittings, it appears that the motors had not been greased in a long time. All motor brushes have only have a small percentage of contact with the rings.

The motors are provided with space heaters. Most covers are missing cover screws and one enclosure is physically damaged. The span motor #1 secondary resistor overheats.

The span drive motors are in good condition. The drives controlled the acceleration, deceleration, running speed and seating very well. It should be noted that the #3 motor/drive system is out of service until the tachometer/speed switch system is placed back in full service.

Span Drive #1 and Span Drive #3 cabinets have non-functional digital position indicators. Span Drive #1 cabinet also has a number of unterminated conductors.

The South leaf brake solenoid exhibits a very low insulation resistance value. It should be cleaned and retested and if still low, should be replaced.

It should be noted that there is only one rotary cam span position limit switch on each leaf. With no redundancy, this is a single point of failure that can stop the operation of the leaf. This switch should be supplemented with a second switch or maintenance should be meticulously performed.

Navigation lights are provided on the fenders and on the leafs. The unit for track #1 is being rehabilitated and track #4 had the bulb out and was missing a cover screw.

The motors/drives being used for operation for testing was North Motor #1 and South Motor #2. Motor #3 was non-functional because the new tachometer-speed switch assembly installation was not completed. Multiple operations took place. The #4 drive faulted on a permissive failure during testing. The problem was not able to be traced at the time of the inspection.

The time of operation of the North Leaf is 1m:33s raise, 2m:12s lower.

5. <u>High Towers</u>

The catenary towers are overall in poor condition. At Catenary towers 862 and 863, all legs have section loss up to ½ inch deep on the interior sides of the angles along the edges of vertical gusset plates at the splices (maximum loss is on catenary tower 863 on the north tower leg at panel point 12 with 32.4% section loss to the southwest angle, the other three angles also have loss at this panel; 27.5% total section loss for all 4 angles), and cover plate loss up to 90%, typical at splices with thin shim plates (maximum loss found on catenary tower 863 on 2 of 4 legs on the north tower leg at panel point 8; 90% section loss to the cover plate, 17.4% section loss to the total section, including the angle; 16.4% total section loss for all 4 angles). The horizontal bracing members at panel points 0-3 have extensive section loss at isolated locations including catenary tower 862, north tower leg, panel point 3, north horizontal member, top angle has 20-foot long x 100% loss to the horizontal leg and 20-foot long by 1 inch high perforation to the vertical leg. The section loss is typically at the member connections with perforations up to 10 inches x 4 inches. The deck plates at panel points 2 & 3 (both tower legs) have laminated rust throughout with section loss and perforations up to 5 feet x 4 feet. The gusset plates at the horizontal member connection typically have section loss up to ⅜ inch deep along the top of the horizontal member. There are isolated gusset plates that have section loss around all sides of the horizontal member.

On catenary tower 862A, the only sizeable section loss to the leg angles is on the tower legs at the base (at the concrete encasement). The worst case is the south tower leg at the upper part of the encasement (panel point 29.5), the southwest angle, south leg has 1/16 inch remaining and the west leg has ⅛ inch remaining x 3 inches high with 1½-inch x ¾ inch edge loss to both angle legs (88.3% section loss to the angle, 25.4% total section loss to all 4 angles). The bracing and lacing bars also have section loss at the concrete encasement interface with up to 100% section loss on isolated members.

The tower leg anchorage assemblies are in fair condition. Catenary tower 862A anchorages are encased in concrete and are not visible. Catenary towers 862 and 863, all stiffener plates have random section loss at the base with as little as ½ inch remaining and edge perforations up to 4 inches high x 2 inches wide. Rivet heads have loss up to 100%.

The catenary bridges are in fair condition. The top and bottom bridge access was limited due to live high voltage electric wires. Catenary towers 862 and 863, horizontal diagonals on the upper and lower bridges have up to 2-inch thick pack rust between the double angle vertical legs and up to ±80% loss to the vertical legs at the lower bridge and perforations up to 3-inch diameter on the upper bridge. The middle bridge vertical gusset plates at the horizontal strut connection typically have up to 1 inch pack rust with 10-inch long x 3-inch high x up to ¼ inch section loss with isolated perforations up to 2 inches x 2 inches around the connection. Catenary tower 863 middle bridge, east truss, bottom flange cover plate near midspan has 10-inch long x 3-inch wide x ¼-inch section loss along the bottom flange angle with a 3-inch x 3-inch perforation at the bottom of the vertical stiffener. The bottom flange rivets at this location have up to 100% head loss. Catenary tower 862A bridges have isolated areas of pack rust up to 1 inch thick at connections, bending the affected members.

The tower foundations are in fair condition. Catenary towers 862 and 863 foundations have severe scale (up to 12 inches deep at edges and corners) over approximately 50% of the visible surfaces with random hollow areas. The exposed concrete is soft. Catenary tower 862, north leg foundation at the northwest corner has scale that undermines the tower base plate up to 2 inches deep. Catenary 862A concrete encasement has areas of light to moderate scale and isolated corner spalls, and the bridge pier stone masonry has approximately 5% loose and missing mortar above the water line.

The anchor bolts are in poor condition. All anchor bolts have laminated rust with section loss at the baseplate with as little as 2⅛" diameter remaining (3" dia. orig., 49.8% section loss). Catenary 862, north tower leg anchorage, west line of bolts has 31.6% total section loss, east line of bolts has 16.0% total section loss; the south tower leg anchorage, west line of bolts has 31.2% total section loss, east line of bolts has 27.0% total section loss. Catenary tower 863, north tower leg anchorage, west line of bolts has 15.9% total section loss, east line of bolts has 16.9% total section loss; the south tower leg anchorage, west line of bolts has 17.9% total section loss, east line of bolts has 15.9% total section loss. Catenary tower 862A anchor bolts are encased in concrete and are not visible.

See Figures IV-1 through IV-4 for Inspection findings at the High Towers.

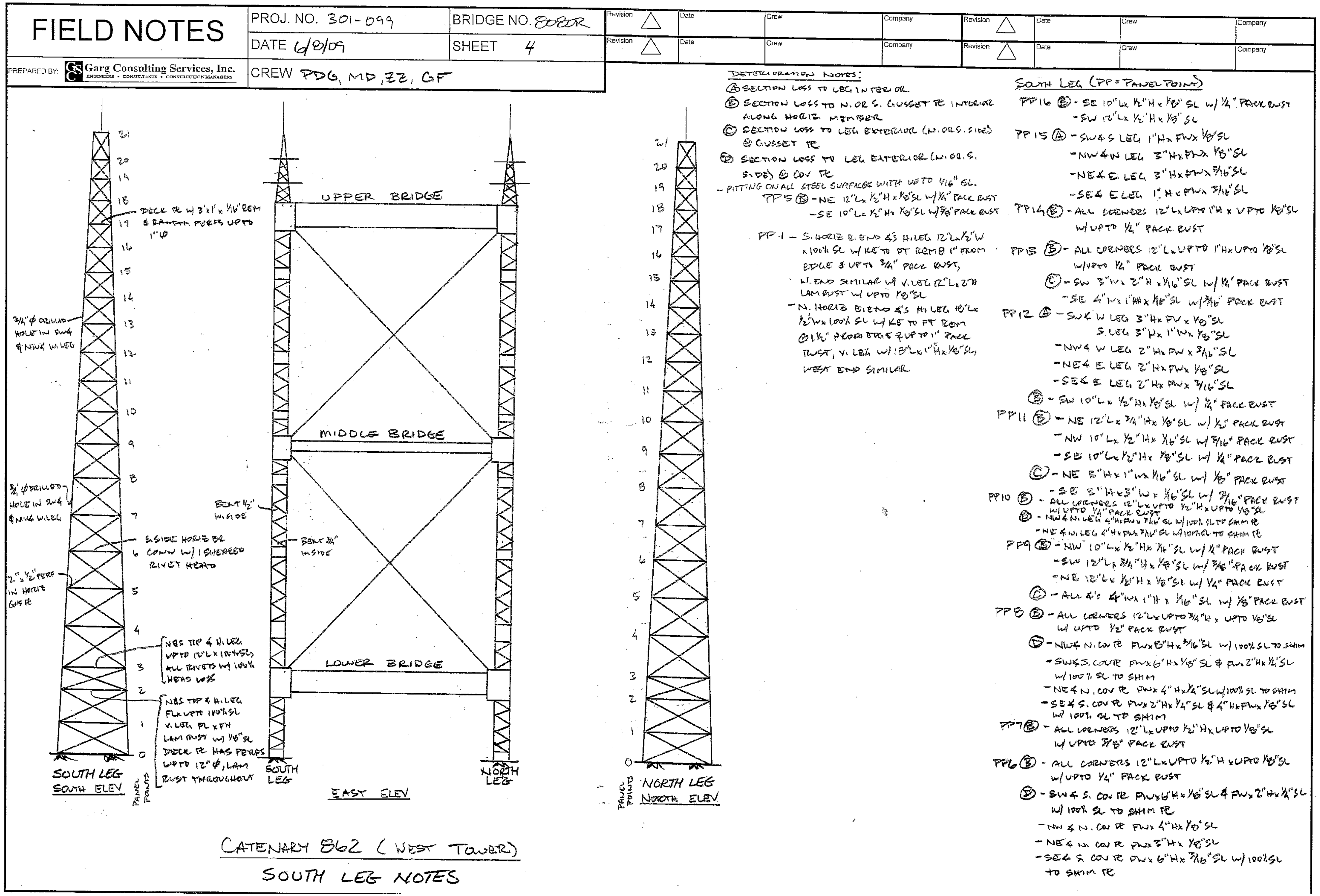


Figure IV-1: High Tower 862 South Leg Inspection Findings

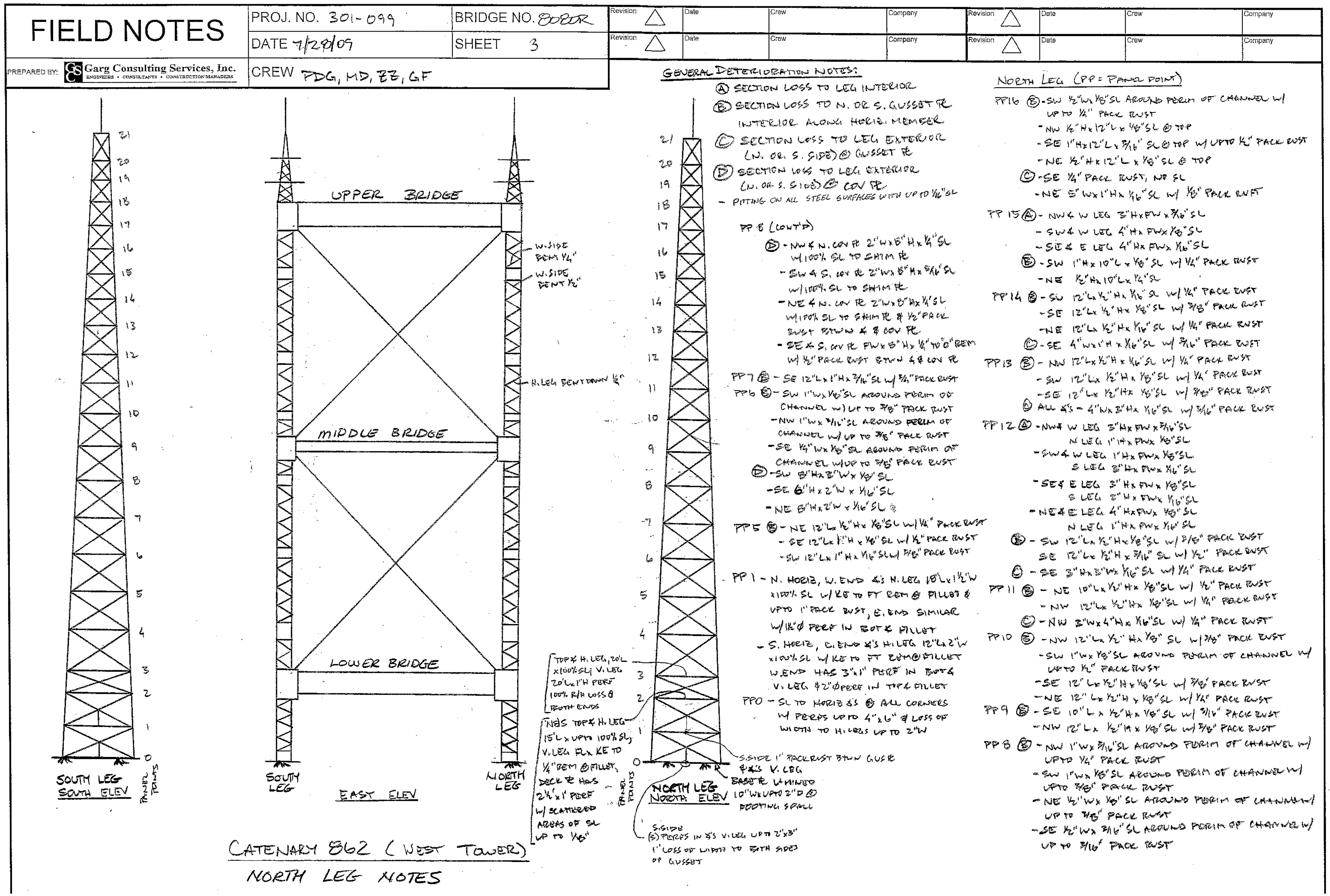


Figure IV-2: High Tower 862 North Leg Inspection Findings

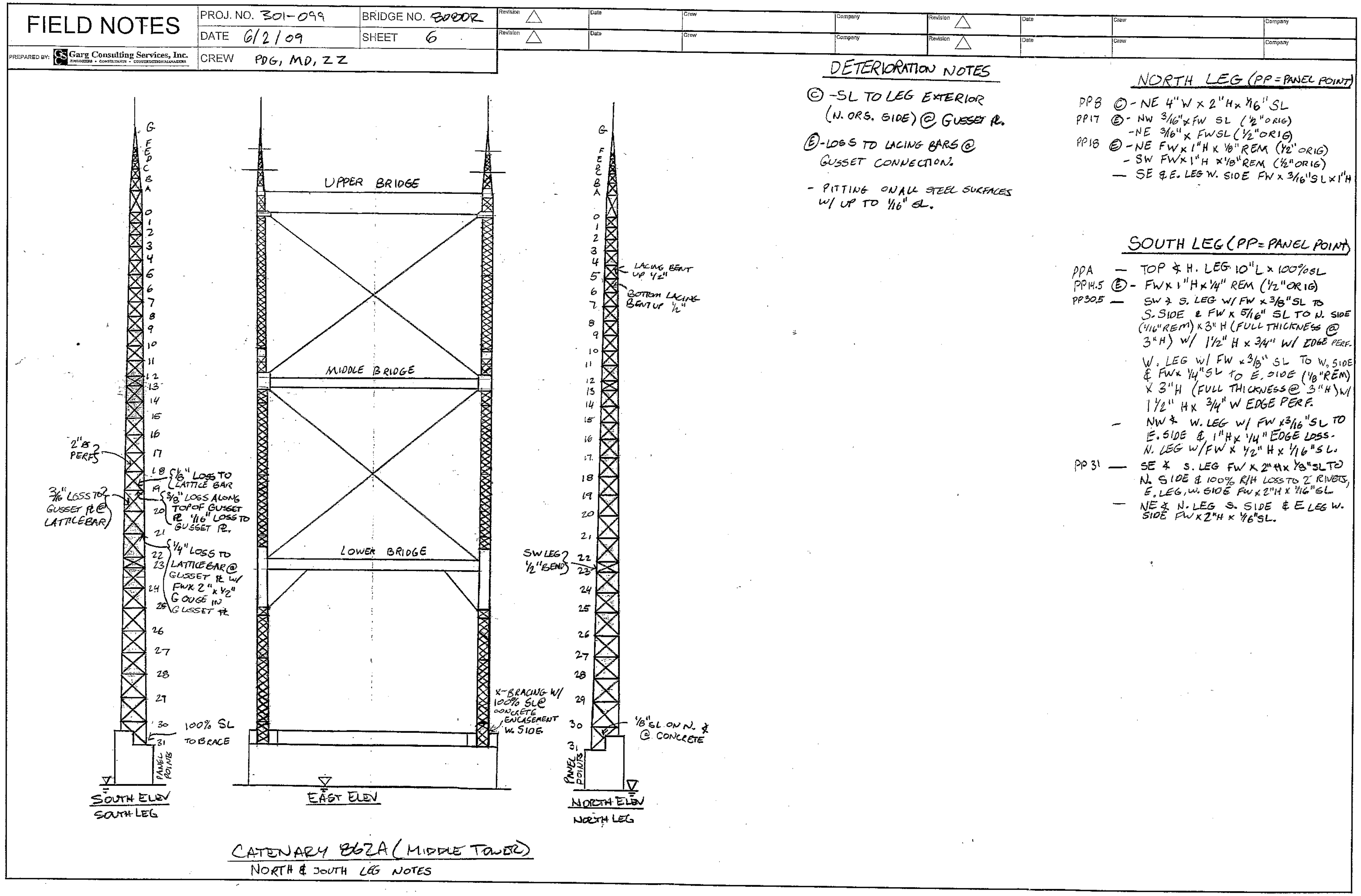


Figure IV-3: High Tower 862A North Leg Inspection Findings

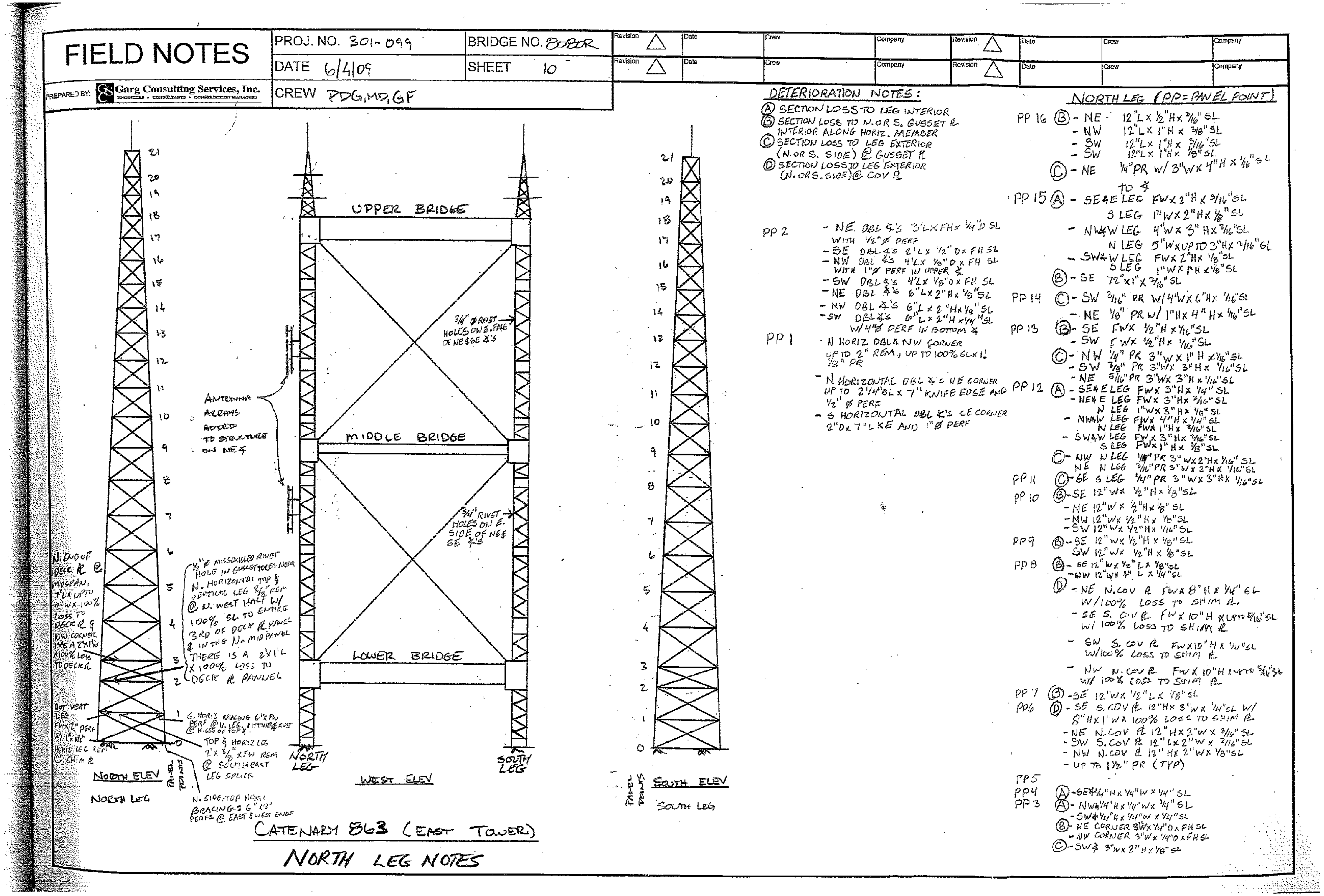


Figure IV-4: High Tower 862 North Leg Inspection Findings

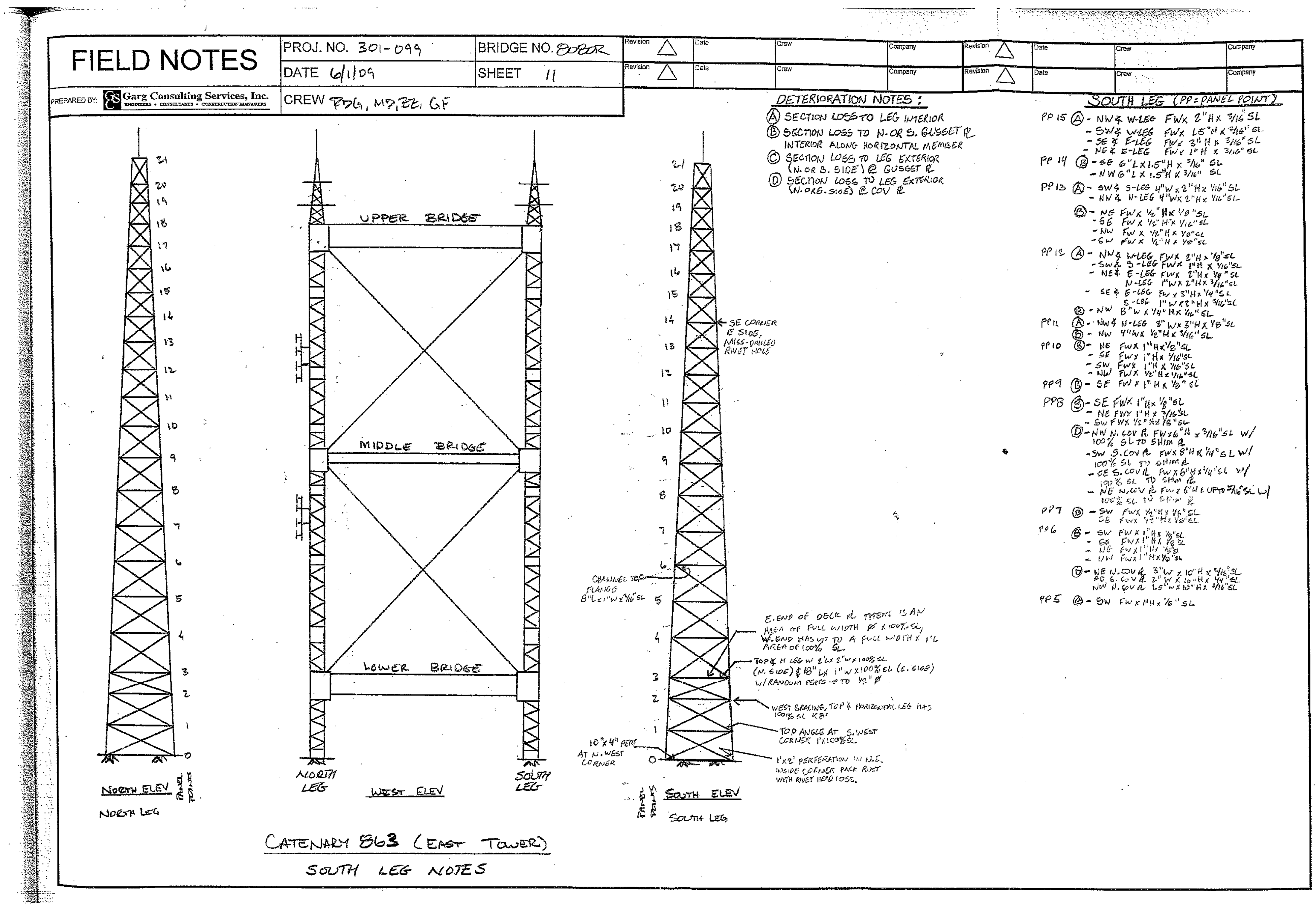


Figure IV-5: High Tower 863 South Leg Inspection Findings

B.) Structural Analysis and Load Ratings

1. Bridge Load Rating

A load rating was conducted in accordance with the 2007 AREMA Specifications, Chapter 15 Section 7.3. As such, both "Normal" and "Maximum" load ratings were developed. As defined by AREMA, the "Normal" rating is the load level which can be carried by the existing structure for its expected service life, while the "Maximum" rating is the load level which the structure can support at infrequent intervals, with any applicable speed restrictions. A "Fatigue" rating analysis was also performed in accordance with AREMA Chapter 15, Section 1.3.13 and Table 15-1-9. Load ratings for all three conditions were prepared for both the "As-built" and "As-inspected" conditions.

The load rating methodology and format used was similar to the 2001 In-Depth Inspection Report prepared by McLaren/DiCesare Consulting Engineers. As is customary with load ratings, only primary load carrying members were analyzed; truss members, girders, etc. Secondary members, such as diaphragms or cross bracing, were not included.

The rating analyses were based on the standard Cooper E-80 loading configuration. Open-hearth steel with a yield stress of 30 ksi was assumed in accordance with AREMA Chapter 15, Section 7.3.4.3. For the Fatigue analysis, fatigue Stress Category "C" was assumed for all members except the truss eyebars, for which Category "E" was assumed.

Member properties are based on as-built drawings from 1905. Cross-sectional areas and stiffness are based on the member and plate sizes from these drawings. Repairs were made to the bridge in 1990. Components of various members were replaced in kind. Span 2 girders, however, had longitudinal stiffener angles replaced with WT sections. These new WT sections were incorporated into the section properties for Span 2 girders for this report.

Loads were calculated based on AREMA for live load, impact, and, in the case of fatigue, mean impact. The AREMA code has been updated since 2001, with differences in mean impact load used in fatigue loading, and member capacities. Truss reactions were calculated using the computer analysis software STAAD Pro, utilizing tension only members and moving loads that spanned the entire bridge. It should be noted that the train loads used in the 2001 load rating did not extend for the entire length of the bridge, nor did it take into account the engine portion of the train leaving the bridge.

Member capacities and ratings were calculated based on AREMA. Differences in versions of AREMA have resulted in differences in capacities. The 2007 version of AREMA has slightly different coefficients for compression members, higher capacities for tension members with respect to the use of the gross cross-sectional area instead of the net area, and slightly different coefficients for the compression side of bending members. Comparison with the 2001 report shows differences in "as-built" Normal and Maximum ratings. Fatigue ratings are higher in the current analysis due to the decrease in mean impact load in the 2007 AREMA Code.

There is little to no critical deterioration to the compression truss members and therefore no as-inspected ratings were necessary for these members. Section loss at the ends of the eyebar ends due to wearing against adjacent members caused reduction in load ratings for several tension members; therefore "as-inspected" ratings were prepared for these members. Floor beams and stringers exhibit various degrees of deterioration. The computer drafting software AutoCAD was used to calculate the section properties of the "as-inspected" sections. The as-built section was drawn and checked against the hand calculated values of area and stiffness. This section was then modified to show section losses such as edge deterioration, thinning, or holes. Values for the properties of the "as-inspected" section are similar to the 2001 report, but could not be exactly correlated because the 2001 report did not explicitly call out the deterioration to each member.

The controlling As-Inspected "Normal" rating for the bridge is Cooper E-50. The controlling "As-Inspected" "Maximum" rating for the bridge is Cooper E-74. The controlling "As-Inspected" "Fatigue" rating for the bridge is Cooper E-45. The controlling member for all three of these ratings is Member U8-M9 (diagonal) of Truss 3 in Span 7, which supports Tracks 2 and 4.

The controlling ratings and governing location, listed by track, are as follows:

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-50	E-84	E-51
	Controlling Member	*Span 4: FB 0 (Track 1&3)*	*Span 7: L6-M7 (Truss 1)*	*Span 7: L6-M7 (Truss 1)*
1	Rating	E-50	E-75	E-51
	Controlling Member	*Span 4: FB 0 (Track 1&3)*	*Span 5: STR 3 (FB5-6)*	*Span 7: L6-M7 (Truss 1)*
2	Rating	E-50	E-74	E-45
	Controlling Member	*Span 7: U8-M9 (Truss 3)*	*Span 7: U8-M9 (Truss 3)*	*Span 7: U8-M9 (Truss 3)*
4	Rating	E-50	E-74	E-45
	Controlling Member	*Span 7: U8-M9 (Truss 3)*	*Span 7: U8-M9 (Truss 3)*	*Span 7: U8-M9 (Truss 3)*

Refer to the following pages for controlling members by span and component.

Span: **1**

Condition: **As-Built**

Member	Controlling Member	Cooper E-80 Rating Normal	Cooper E-80 Rating Maximum	Cooper E-80 Rating Fatigue
Truss	Diagonal L0-M1	E-104		
	Diagonal L0-M1		**E-159**	
	Diagonal M5-U4			**E-87**
Floorbeam	Interior Floorbeam @ End	E-115		
	Interior Floorbeam @ End		E-201	
	Interior Floorbeam @ Midspan			E-97
Stringer	Typical Stringer @ End	**E-100**		
	Typical Stringer @ End		E-173	
	Typical Stringer @ Midspan			E-91
	Controlling Rating	**E-100**	**E-159**	**E-87**

Condition: **As-Inspected**

Member	Controlling Member	Cooper E-80 Rating Normal	Cooper E-80 Rating Maximum	Cooper E-80 Rating Fatigue
Truss*	Diagonal L0-M1	E-104		
	Diagonal L0-M1		E-159	
	Diagonal M5-U4			E-87
Floorbeam	FB 11 (Track 2&4) @ Midspan	E-102		
	FB 11 (Track 2&4) @ Midspan		E-150	
	FB 11 (Track 2&4) @ Midspan			E-84
Stringer	STR 3 (Track 1, FB 8 - FB 9) Midspan	**E-57**		
	STR 3 (Track 1, FB 8 - FB 9) Midspan		**E-84**	
	STR 3 (Track 1, FB 9 - FB 10) Midspan			**E-83**
	Controlling Rating	**E-57**	**E-84**	**E-83**

* No critical loss in truss members based on inspection findings

Condition: **As-Inspected (by Track)**

Track		Cooper E-80 Rating Normal	Cooper E-80 Rating Maximum	Cooper E-80 Rating Fatigue
3	Rating	E-104	E-117	E-86
	Controlling Member	*L0-M1 (Typ. Truss)*	*FB 5 (Track 1&3)*	*FB 5 (Track 1&3)*
1	Rating	E-57	E-84	E-83
	Controlling Member	*STR 3 (FB8-9)*	*STR 3 (FB8-9)*	*STR 3 (FB8-9)*
2	Rating	E-102	E-150	E-84
	Controlling Member	*FB 11 (Track 2&4)*	*FB 11 (Track 2&4)*	*FB 11 (Track 2&4)*
4	Rating	E-102	E-150	E-84
	Controlling Member	*FB 11 (Track 2&4)*	*FB 11 (Track 2&4)*	*FB 11 (Track 2&4)*

Span: **2**

Condition: **As-Built**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Girder	Moment @ Midspan	**E-105**		
	Moment @ Midspan		**E-159**	
	Moment @ Midspan			**E-115**
Floorbeam	N/A			
	N/A			
	N/A			
Stringer	N/A			
	N/A			
	N/A			
	Controlling Rating	**E-105**	**E-159**	**E-115**

Condition: **As-Inspected**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Girder	G1 Moment @ Midspan	**E-104**		
	G1 Moment @ Midspan		**E-159**	
	G1 Moment @ Midspan			**E-114**
Floorbeam	N/A			
	N/A			
	N/A			
Stringer	N/A			
	N/A			
	N/A			
	Controlling Rating	**E-104**	**E-159**	**E-114**

Condition: **As-Inspected (by Track)**

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-104	E-115	E-114
	Controlling Member	*G1*	*G1*	*G1*
1	Rating	E-105	E-159	E-115
	Controlling Member	*G3/G4*	*G3/G4*	*G3/G4*
2	Rating	E-105	E-150	E-84
	Controlling Member	*G5/G6*	*G5/G6*	*G5/G6*
4	Rating	E-105	E-150	E-84
	Controlling Member	*G7/G8*	*G7/G8*	*G7/G8*

Span: 3

Condition: As-Built

Member	Controlling Member	Cooper E-80 Rating Normal	Maximum	Fatigue
Girder	Controlled by Dead Load (Bridge Open)			
	Controlled by Dead Load (Bridge Open)			
	Controlled by Dead Load (Bridge Open)			
Floorbeam	Middle Floorbeam @ 9.83'	**E-80**		
	Middle Floorbeam @ 9.83'		**E-119**	
	Middle Floorbeam @ 9.83'			**E-82**
Stringer	Typical Stringer @ Midspan	E-114		
	Typical Stringer @ Midspan		E-167	
	Typical Stringer @ Midspan			E-95
	Controlling Rating	**E-80**	**E-119**	**E-82**

Condition: As-Inspected

Member	Controlling Member	Cooper E-80 Rating Normal	Maximum	Fatigue
Girder	Controlled by Dead Load (Bridge Open)			
	Controlled by Dead Load (Bridge Open)			
	Controlled by Dead Load (Bridge Open)			
Floorbeam	FB 1 (Track 1&3) @ 9.83'	**E-79**		
	FB 1 (Track 1&3) @ 9.83'		**E-117**	
	FB 1 (Track 1&3) @ 9.83'			**E-80**
Stringer	STR 6 (Track 2, FB 1 - FB 2) @ End	E-97		
	STR 2 (Track 3, FB 0 - FB 1) @ Midspan		E-165	
	STR 2 (Track 3, FB 0 - FB 1) @ Midspan			E-98
	Controlling Rating	**E-79**	**E-117**	**E-80**

Condition: As-Inspected (by Track)

Track		Cooper E-80 Rating Normal	Maximum	Fatigue
3	Rating	E-79	E-117	E-80
	Controlling Member	*FB 1 (Track 1&3)*	*FB 1 (Track 1&3)*	*FB 1 (Track 1&3)*
1	Rating	E-79	E-117	E-80
	Controlling Member	*FB 1 (Track 1&3)*	*FB 1 (Track 1&3)*	*FB 1 (Track 1&3)*
2	Rating	E-97	E-167	E-95
	Controlling Member	*STR 6 (FB1-2)*	*Typical Stringer*	*Typical Stringer*
4	Rating	E-114	E-167	E-95
	Controlling Member	*Typical Stringer*	*Typical Stringer*	*Typical Stringer*

Span: **4**

Condition: **As-Built**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Top Chord U2-U3	E-90		
	Top Chord U2-U3		**E-127**	
	Bottom Chord L1-L2			**E-68**
Floorbeam	Interior Floorbeam @ End	**E-76**		
	Interior Floorbeam @ End		E-133	
	Interior Floorbeam @ Midspan			E-101
Stringer	Typical Stringer @ Midspan	E-109		
	Typical Stringer @ Midspan		E-160	
	Typical Stringer @ Midspan			E-95
	Controlling Rating	**E-76**	**E-127**	**E-68**

Condition: **As-Inspected**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss*	Top Chord U2-U3	E-90		
	Top Chord U2-U3		E-127	
	Bottom Chord L1-L2			**E-68**
Floorbeam	FB 0 (Track 1&3) @ Midspan	**E-50**		
	FB 0 (Track 1&3) @ Midspan		**E-89**	
	FB 4 (Track 2&4) @ Midspan			E-97
Stringer	STR 4 (FB 1 – FB 2) @ Midspan	E-52		
	STR 4 (FB 1 – FB 2) @ Midspan		E-131	
	STR 2 (FB 4 – FB 5) @ Midspan			E-85
	Controlling Rating	**E-50**	**E-89**	**E-68**

* No critical loss in truss members based on inspection findings

Condition: **As-Inspected (by Track)**

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-50	E-89	E-68
	Controlling Member	*FB 0 (Track 1&3)*	*FB 0 (Track 1&3)*	*L1-L2 (Typ. Truss)*
1	Rating	E-50	E-89	E-68
	Controlling Member	*FB 0 (Track 1&3)*	*FB 0 (Track 1&3)*	*L1-L2 (Typ. Truss)*
2	Rating	E-75	E-127	E-68
	Controlling Member	*FB 2 (Track 2&4)*	*U2-U3 (Typ. Truss)*	*L1-L2 (Typ. Truss)*
4	Rating	E-75	E-127	E-68
	Controlling Member	*FB 2 (Track 2&4)*	*U2-U3 (Typ. Truss)*	*L1-L2 (Typ. Truss)*

Span: 5

Condition: As-Built

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Diagonal L4-M5	**E-64**		
	Diagonal L4-M5		**E-92**	
	Diagonal L4-M5			**E-54**
Floorbeam	Interior Floorbeam @ Midspan	E-102		
	Interior Floorbeam @ Midspan		E-150	
	Interior Floorbeam @ Midspan			E-108
Stringer	Typical Stringer @ Midspan	E-110		
	Typical Stringer @ Midspan		E-162	
	Typical Stringer @ Midspan			E-94
	Controlling Rating	**E-64**	**E-92**	**E-54**

Condition: As-Inspected

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Diagonal M9-U10 (Truss 3, Track 2-4)	E-56		
	Diagonal M9-U10 (Truss 3, Track 2-4)		E-88	
	As-Built*: Diagonal L4-M5			**E-54**
Floorbeam	FB 1 (Track 1&3) @ Midspan	E-93		
	FB 1 (Track 1&3) @ Midspan		E-137	
	FB 1 (Track 1&3) @ Midspan			E-99
Stringer	STR 3 (Track 1, FB 5 - FB 6) @ Midspan	**E-51**		
	STR 3 (Track 1, FB 5 - FB 6) @ Midspan		**E-75**	
	STR 3 (Track 1, FB 5 - FB 6) @ Midspan			E-76
	Controlling Rating	**E-51**	**E-75**	**E-54**

* No critical loss in truss members based on inspection findings

Condition: As-Inspected (by Track)

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-64	E-92	E-54
	Controlling Member	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*
1	Rating	E-51	E-75	E-54
	Controlling Member	*STR 3 (FB5-6)*	*STR 3 (FB5-6)*	*L4-M5 (Typ. Truss)*
2	Rating	E-56	E-88	E-54
	Controlling Member	*M9-U10 (Truss 3)*	*M9-U10 (Truss 3)*	*L4-M5 (Typ. Truss)*
4	Rating	E-56	E-88	E-54
	Controlling Member	*M9-U10 (Truss 3)*	*M9-U10 (Truss 3)*	*L4-M5 (Typ. Truss)*

Span: **6**

Condition: **As-Built**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Diagonal L4-M5	**E-64**		
	Diagonal L4-M5		**E-92**	
	Diagonal L4-M5			**E-54**
Floorbeam	Interior Floorbeam @ Midspan	E-102		
	Interior Floorbeam @ Midspan		E-150	
	Interior Floorbeam @ Midspan			E-108
Stringer	Typical Stringer @ Midspan	E-110		
	Typical Stringer @ Midspan		E-162	
	Typical Stringer @ Midspan			E-94
	Controlling Rating	**E-64**	**E-92**	**E-54**

Condition: **As-Inspected**

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Bot. Chord L10-L12 (Truss 3, Track 2-4)	**E-62**		
	As-Built*: Diagonal L4-M5		**E-92**	
	As-Built*: Diagonal L4-M5			**E-54**
Floorbeam	FB 1 (Tr 2&4) @ Midspan	E-97		
	FB 1 (Tr 2&4) @ Midspan		E-143	
	FB 1 (Tr 2&4) @ Midspan			E-104
Stringer	STR 3 (FB 2 - FB 3) @ Midspan	E-97		
	STR 3 (FB 2 - FB 3) @ Midspan		E-142	
	STR 3 (FB 2 - FB 3) @ Midspan			E-86
	Controlling Rating	**E-62**	**E-92**	**E-54**

* No critical loss in truss members based on inspection findings

Condition: **As-Inspected (by Track)**

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-64	E-92	E-54
	Controlling Member	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*
1	Rating	E-64	E-92	E-54
	Controlling Member	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*
2	Rating	E-62	E-92	E-54
	Controlling Member	*L10-L12 (Truss 3)*	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*
4	Rating	E-62	E-92	E-54
	Controlling Member	*L10-L12 (Truss 3)*	*L4-M5 (Typ. Truss)*	*L4-M5 (Typ. Truss)*

Span: 7

Condition: As-Built

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Diagonal L4-M5	**E-64**		
	Diagonal L4-M5		**E-92**	
	Diagonal L4-M5			**E-54**
Floorbeam	Interior Floorbeam @ Midspan	E-102		
	Interior Floorbeam @ Midspan		E-150	
	Interior Floorbeam @ Midspan			E-108
Stringer	Typical Stringer @ Midspan	E-110		
	Typical Stringer @ Midspan		E-162	
	Typical Stringer @ Midspan			E-94
	Controlling Rating	**E-64**	**E-92**	**E-54**

Condition: As-Inspected

		Cooper E-80 Rating		
Member	Controlling Member	Normal	Maximum	Fatigue
Truss	Diagonal U8-M9 (Truss 3, Track 2-4)	**E-50**		
	Diagonal U8-M9 (Truss 3, Track 2-4)		**E-74**	
	Diagonal U8-M9 (Truss 3, Track 2-4)			**E-45**
Floorbeam	FB 7 (Track 2&4)	E-94		
	FB 7 (Track 2&4)		E-150	
	FB 7 (Track 2&4)			E-100
Stringer	STR 4 (Track 1, FB 2- FB 3)	E-78		
	STR 4 (Track 1, FB 2- FB 3)		E-115	
	STR 4 (Track 1, FB 2- FB 3)			E-76
	Controlling Rating	**E-50**	**E-74**	**E-45**

Condition: As-Inspected (by Track)

		Cooper E-80 Rating		
Track		Normal	Maximum	Fatigue
3	Rating	E-58	E-84	E-51
	Controlling Member	*L6-M7 (Truss 1)*	*L6-M7 (Truss 1)*	*L6-M7 (Truss 1)*
1	Rating	E-58	E-84	E-51
	Controlling Member	*L6-M7 (Truss 1)*	*L6-M7 (Truss 1)*	*L6-M7 (Truss 1)*
2	Rating	E-50	E-74	E-45
	Controlling Member	*U8-M9 (Truss 3)*	*U8-M9 (Truss 3)*	*U8-M9 (Truss 3)*
4	Rating	E-50	E-74	E-45
	Controlling Member	*U8-M9 (Truss 3)*	*U8-M9 (Truss 3)*	*U8-M9 (Truss 3)*

2. High Tower Analysis

The as-built structural analysis for 862 and 863 shows that there are many secondary members that are overloaded, especially at the tops of the towers below the extension for the ground wires. Overstressing of members occurred under all load cases, NESC Load Cases 250B (Combined Ice and Wind Loading), 250C (Extreme Wind Loading), and 250D (Extreme Ice and Wind Loading). For the as-inspected results, each high tower had the six load cases applied (wind in two directions for each NESC case, a total of 12 cases).

The as-built structural analysis for 862A shows many tower legs and secondary members that are overloaded, especially at the tops of the towers below the extension for the ground wires and at the tower base below the lower bridge. Overstressing of members occurred under all load cases. For the as-inspected results, the high tower had six load cases applied.

Many of the secondary bracing and lacing bars are slender members with Kl/r ratios greater than 200 and are not designed to handle compressive forces. These are reflected in the results.

<u>As-built case 250B (Ice and Wind combined) loading for 862 and 863:</u>

The tower legs have primary members that are overstressed near the middle bridge in panels 9 and 10. Note that nearly all members, primary and secondary, in panel 21 for both legs (where the ground wire extensions are attached) are overstressed. The vertical bracing on the north and south faces of both legs is typically overstressed in panels 4 through 14. Isolated vertical bracing on the east and west faces at the tower base in panels 1 through 3 are overstressed.

The sway brace main members on the crossing span side are overstressed in compression and isolated main members on the side span side are overstressed in tension.

<u>As-built case 250C (Extreme Wind) loading for 862 and 863:</u>

The tower leg primary members are overstressed in panels 9 and 10, similarly to case 250B. In addition, isolated primary members at the base in panel 1 are overstressed. Nearly all members, primary and secondary, in panel 21 for both legs (where the ground wire extensions are attached) are overstressed. The vertical bracing on the north and south faces of both legs is overstressed similarly to Case 250B.

The sway braces in this case are overstressed similarly to the results of the 250B load case, but not as severe.

<u>As-built case 250D (Extreme Ice and Wind) loading for 862 and 863:</u>

There are no tower leg primary members that are overstressed. Nearly all members, primary and secondary, in panel 21 for both legs (where the ground wire extensions are attached) are overstressed. Isolated vertical bracing in panels 4-9 are overstressed.

The sway braces in this case are overstressed similarly to the results of the 250B load case, but not as severe.

<u>As-built case 250B (Ice and Wind combined) loading for 862A:</u>

The tower legs have isolated primary members that are overstressed above the middle bridge in panels 11 and 12 (pending full determination of location). The vertical bracing on the north and south faces of both legs is typically overstressed in panels A through G and 1 through 23 (every panel above the lower bridge). The upper bridge has two isolated chord members that are overstressed near midspan, and scattered vertical members near the tower legs that are overstressed.

The sway brace main members on the crossing span side are overstressed in compression and isolated main members on the side span side are overstressed in tension.

As-built case 250C (Extreme Wind combined) loading for 862A:

There are widespread overstressed members, especially in the bridges and tower leg bracing. The tower legs have primary members that are overstressed near the bridges in panels B, 1, 12, 14, and 22 (pending full determination of location) and at the base in panels 30 and 31. The vertical bracing on the north and south faces of both legs is typically overstressed in panels A through G and 1 through 23 (every panel above the lower bridge). All three bridges have chord members that are overstressed throughout, and all vertical members in the lower bridge are overstressed.

The sway brace main members on the crossing span side are overstressed in compression and isolated main members on the side span side are overstressed in tension.

As-built case 250D (Extreme Ice and Wind combined) loading for 862A:

The tower legs have primary members throughout that are overstressed except below the lower bridge in panels 24 through 28. The vertical bracing on the north and south faces of both legs is typically overstressed in panels A through G and 1 through 23 (every panel above the lower bridge). The upper bridge top chord on the crossing span side has widespread members that are overstressed and the bottom chords near midspan have members that are overstressed. There are isolated vertical members near the tower legs that are overstressed.

The sway brace main members on the crossing span side are overstressed in compression and isolated main members on the side span side are overstressed in tension.

As-Inspected Analysis

A structural analysis was performed for all members in High Towers 862, 862A, and 863 with significant section loss. Significant section loss is defined as primary members with greater than 5% section loss and secondary members (bracing) with greater than 50% section loss. All other loss was considered negligible for analysis results.

All members that were previously overstressed in the as-built condition remained overstressed. Typically, members with section loss are overstressed, most notably at the base of 862A where there is section loss as high as 80%. The section loss did not affect members adjacent to the areas of section loss.

The secondary members with section loss typically no longer met Kl/r specifications due to the altered section properties caused by the section loss. The section loss to these members only affected the members themselves and did not affect the overall structure.

The negligible affect of the section loss to the structure is an indication that replacement of the structures should be favored over rehabilitation.

A preliminary analysis indicates that the current high towers become overstressed at approximately 50-60 mph based on current design codes.

V. Railroad Operations

A.) Existing Operations

1. General

The Devon Bridge is located within the Metro North New Haven Line, which operates from New Haven, CT to Woodlawn, New York, NY. In Woodlawn, the rail joins the Harlem Line and continues into Manhattan to the Grand Central Station terminus. Three branch lines split from the main line as follows:

New Canaan Branch:	Diverts in Stamford and extends to New Canaan
Danbury Branch:	Diverts in South Norwalk and extends to Danbury
Waterbury Branch:	Diverts in Milford immediately east of the Devon Bridge and extends to Danbury

Passenger service is provided by Metro North Commuter Railroad and National Railroad Passenger Corporation (AMTRAK) trains, while freight service is provided by CSX Transportation. The State of Connecticut is the owner of the line, while it is operated and maintained by Metro North Railroad. The rail line is also part of as Amtrak's Northeast Corridor and is the subject of Amtrak's current investment into high speed rail along the corridor.

2. Service

Metro North operates daily commuter service on the New Haven Line, which serves thirty (30) main stations, nineteen (19) of which are in Connecticut. At the Devon Bridge, Metro North operates approximately 55 eastbound and 52 westbound through trains during weekdays, and 32 eastbound and 31 westbound on weekends. Of these trains, 7 eastbound and 8 westbound trains bound for the Waterbury branches use the bridge on typical weekdays, and six in each direction on weekends. Eastbound trains are sometimes referred to as "Outbound", and westbound trains as "Inbound".

AMTRAK operates intercity trains on the New Haven Line to provide regional service between Boston, Providence, and New York City. AMTRAK operates approximately 22 trains daily in each direction across the bridge on weekends, and 20 on weekdays.

There are 1 to 2 freight trains that utilize the bridge on a daily basis.

3. Motive Power

The majority of MNR trains utilizing the New Haven Line are powered by electric motors. Motive power for these trains is provided by either an overhead catenary or a third rail system. The overhead catenary system is comprised of a contact wire above each track supported from above by a catenary, or messenger, wire. The messenger wire is in turn supported at intervals by a steel frame or bridge mounted structural support. A pantograph mounted atop the train cars runs along the contact wire and distributes power from the wire to the train car.

The third rail system consists of a tertiary rail mounted alongside of the typical rail tracks, and a contact block, known as a shoe, attached to the train cars. Electrical power is distributed from the third rail to the shoe, which runs in contact with, and under, the third rail. The catenary system provides AC power to the train, while the third rail system provides DC power.

Inbound Metro North trains from New Haven utilize a catenary system for power until reaching Pelham, when the third rail system is used to deliver power. Similarly, Amtrak uses the catenary system, while freight uses diesel locomotives.

4. Control

The Devon Bridge is located between CP 257 (CENTRAL) and CP 261 (DEVON).

5. Track Geometry

The west approach to the bridge consists of a tangent section leading out of the Stratford Station toward the bridge. After a slight right turn of 0°22′, the tracks lead into a horizontal curve of 2°03′ with a superelevation of 5¼″. This curve transitions into another horizontally tangent section across the bridge into Milford. The tracks are not superelevated across the bridge. There are two slight curves, with accompanying superelevation, immediately east of the bridge before transitioning to another tangent section.

The New Haven Line generally is climbing from west to east in the vicinity of the Devon Bridge. An approach grade of +0.44% is present leading up to the bridge, which then becomes level as the tracks cross the bridge. The tracks then begin ascending again to a grade of +0.61% as they progress east.

The maximum design speed for trains across the Devon Bridge is 40 mph, with approach speed limits of 60 mph.

The Devon Bridge is located just west of where the Waterbury Branch begins. The Waterbury Branch connects to Track 3 in a Y-configuration to allow for mainline trains from both directions to enter the Waterbury Branch, or for Waterbury Branch trains to enter the mainline in either direction. The westerly leg of the "Y" connection terminates approximately 125 feet from the start of the Devon Bridge. To accommodate the Waterbury Branch trains, a universal interlocking is present across all 4 tracks immediately west of the bridge, and again just east of the easterly leg of the Waterbury Branch Y connection. In addition, traveling eastbound and approximately 4 miles past the bridge, Track 3 terminates into Track 1 at MP 61.

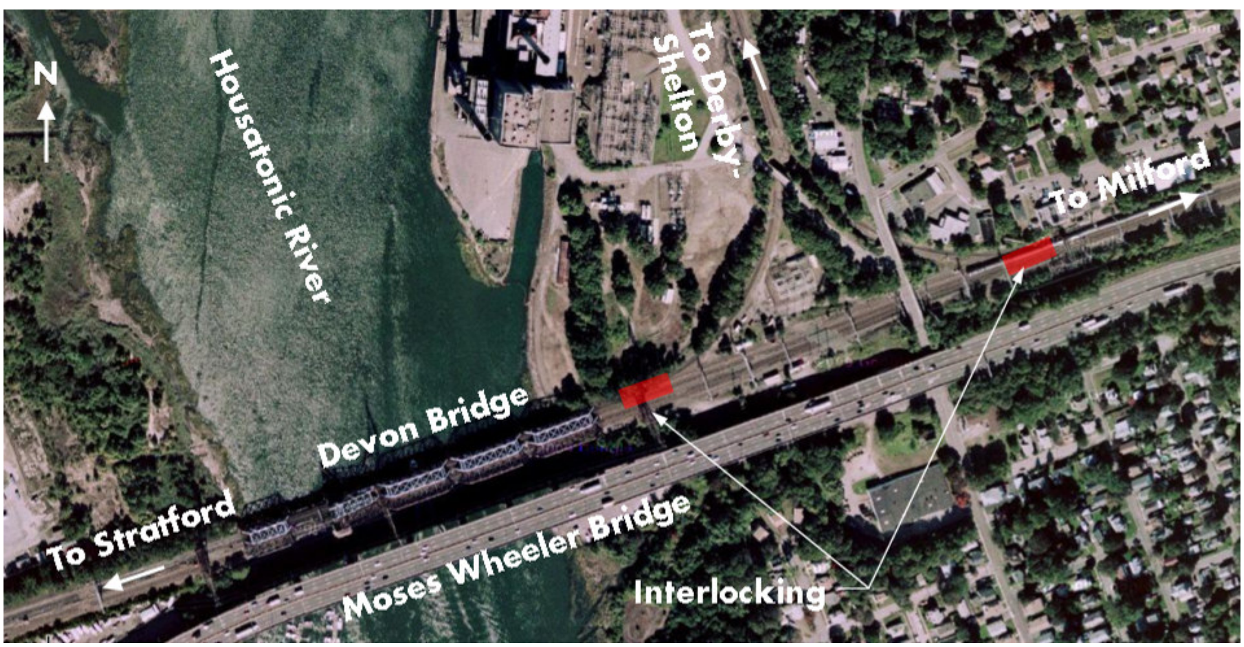


Photo V-1: Aerial View of Bridge

6. Stations

Stratford Station is located approximately 1.4 miles west of the Devon Bridge, and consists of high level station platforms adjacent to the outside tracks (Track 3 for westbound, and Track 4 for eastbound). To the east of the bridge, Milford Station is located approximately 3 miles east of the bridge. This station also consists of high level station platforms adjacent to the outside tracks; however the westbound platform is adjacent to Track 1, as Track 3 terminates as noted previously.

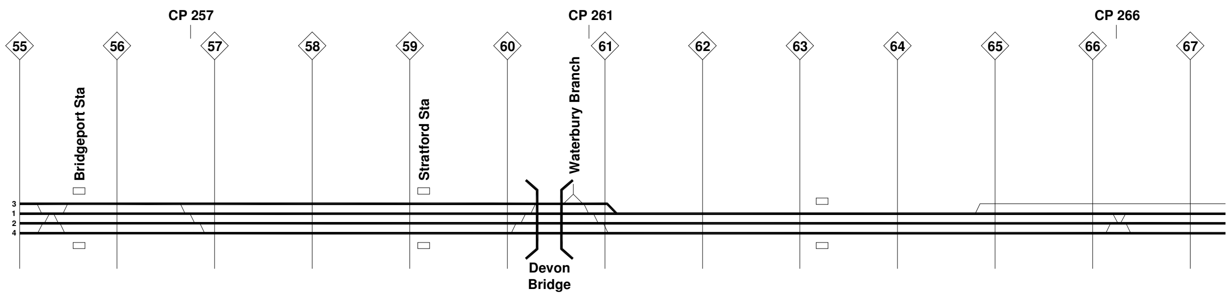


Figure V-1: Track Chart in vicinity of Devon Bridge

B.) Operations During Construction

The Devon Bridge consists of two parallel structures, with the superstructure of each independent of the other and each supporting two tracks. This configuration provides for a convenient way to divide proposed work into two separate stages, whereby one half of the bridge (one structure) is taken out of service during work on that particular half.

The two alternatives presented here utilize the staged construction concept, and provide for two major stages of construction. These alternatives are driven primarily by the location of existing interlocking and the presence of the Waterbury Branch immediately to the east of the bridge. Both of the alternatives facilitate reconstruction of the northerly half of the bridge in the first stage. This is desirable to allow for construction of a new Operator's House to the north of the bridge while the existing Operator's House to the south remains in service.

Staging Comparison		
Staging Alternative	Advantages	Disadvantages
A	Reduces converging train moves through work area Minimizes two track service length (4 miles)	Increases construction cost and complication due to "cut and throw" and temporary connection to Waterbury Branch
B	Does not require modifications to existing track configuration	Requires westbound Waterbury Branch trains to travel east to interlocking before reversing direction to continue west Long two track service length (9 miles) unless "cut and throw" is added

1. Staging Considerations

Both staging alternatives presented herein will provide for adequate room for a major rehabilitation or bridge replacement to occur within the footprint of the existing bridge. Staging Alternative A offers a staging configuration that reduces the converging train moves through the work area. However this comes at the added cost and complication associated with constructing a temporary connection from the Waterbury Branch to Track 2 as well as installing a "cut and throw" during Stage 1. Staging Alternative B could be implemented without the need for temporary or permanent modifications to the existing plant, but will complicate integrating Waterbury Branch trains into the New Haven Line. Both alternatives require temporary bridge plates to accommodate boarding of passengers.

A third staging alternative, Alternative C, would consist of eliminating the Waterbury Branch connection to the New Haven Line, and providing bus service between the Derby-Shelton Station and the Stratford Station. This alternative was dismissed as a viable alternative due to the long-term inconvenience to passengers using the Waterbury Branch.

Based on preliminary input from MNR personnel, Staging Alternative B is the preferred alternative due to the ability to accomplish the staging without the need to realign the Waterbury Branch to Track 2 during construction.

2. Staging Alternative A

The original design drawings for the current Devon Bridge indicate that the Waterbury Branch at one time connected to Tracks 2 and 4 as opposed to Track 3. This alternative would involve recreating this connection to allow westbound trains from the Waterbury Branch cross the Housatonic River while Tracks 1 and 3 are out of service. This alternative requires four (4) miles of two track service during Stage 1, and one (1) mile of two track service during Stage 2.

Stage 1

Stage 1 would be split into two substages to minimize the long-term impacts to rail traffic at the bridge.

Stage 1a involves a track outage of Track 2 to facilitate a new crossover segment on Track 2 in anticipation of the Stage 1b track configuration, when the Waterbury Branch is connected directly to Track 2 east of the bridge. In addition, the Track 1 portion of a "cut and throw" between Tracks 1 and 2 would begin during this stage. With this configuration in place, westbound trains would be switched to Track 1 or Track 3 at MP 61 or CP 261 east of the bridge, and either remain on Track 3 or return to Track 1 at the interlocking west of the bridge. Eastbound trains would be shifted to Track 4 at the CP 257 interlocking west of the Stratford Station, and would remain on Track 4 until CP 266 east of the Milford Station. Eastbound trains could conceivably be switched to Track 2 just east of the bridge at CP 257 if required by switching across Track 2 to Track 1 just west of the bridge and then back to Track 2 east of the bridge. Trains from the Waterbury Branch would maintain their existing routing to and from Track 3. No stations would be affected during this substage.

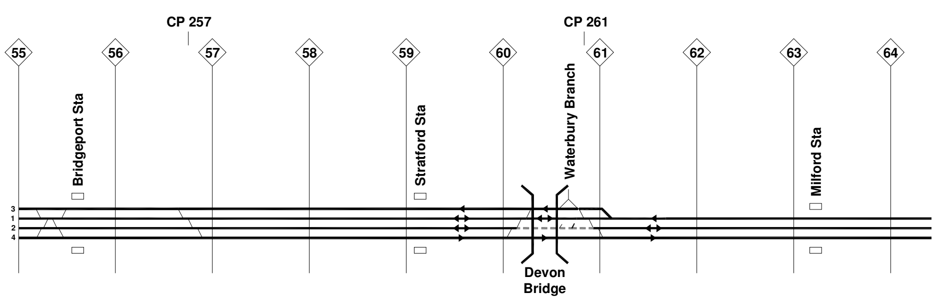


Figure V-2: Staging Alternative A – Stage 1A

Stage 1b would see a long-term track outage between CP 257 and CP 261 at Track 1 and Track 3. During this substage, the "cut and throw" between Tracks 1 and 2 would be completed and the new interlocking would be extended across Track 1 and Track 3 to the westerly leg of the Waterbury Branch "Y" connection. Westbound trains would be switched to Track 2 at the new "cut and throw" east of the bridge, and remain on Track 2 until CP 257 west of the Stratford Station. Eastbound trains would switch to Track 4 at CP 257. Trains from the Waterbury Branch would maintain their existing routing to the east, but would now connect directly to Track 2 during this substage. The Stratford Station would be affected during this substage. Bridge plates across Tracks 1 and 3, or a temporary platform would be required to facilitate inbound passengers boarding the trains on Track 2. During the Stage 1b long-term track outage, the northerly half of the bridge would be rehabilitated or replaced. Upon conclusion of this stage, the "cut and throw" and new temporary connection to the Waterbury Branch across Tracks 1 and 3 would be discontinued and removed which again would require disruption to Waterbury Branch riders.

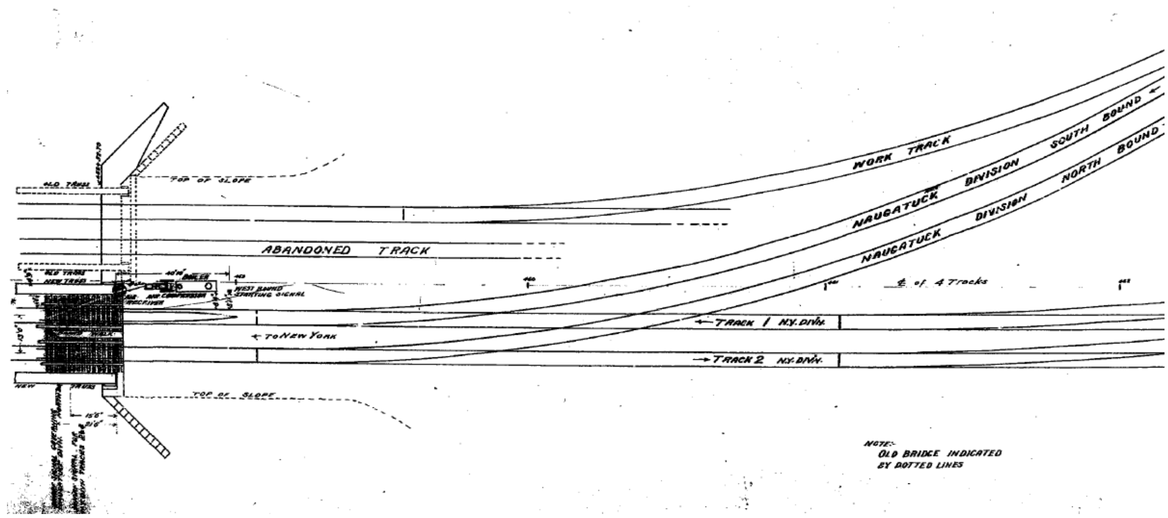


Figure V-2: Track configuration east of bridge, during construction of existing c. 1905

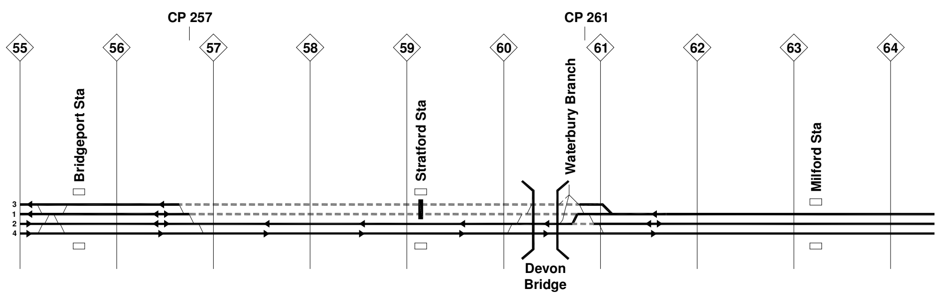


Figure V-3: Staging Alternative A – Stage 1B

Stage 2

Stage 2 involves a long-term track outage at CP 261 of Tracks 2 and 4 to facilitate construction relating to the southerly half of the bridge. The presence of the universal interlocking immediately adjacent to the bridge permits this staging configuration to be implemented relatively easily. Just over one mile of track is affected by this configuration, and there is no affect at any stations. Westbound trains would be switched to Track 3 at MP 61 east of the bridge, and either remain on Track 3 or return to Track 1 at the interlocking west of the bridge. Eastbound trains would be shifted to Track 1 at the CP 261 interlocking just west of the bridge, and would return to either Track 2 or Track 4 at the interlocking east of the bridge. Trains from the Waterbury Branch would maintain their original routing to and from Track 3. During the Stage 2 permanent track outage, the southerly half of the bridge would be rehabilitated or replaced. No stations would be affected during this stage.

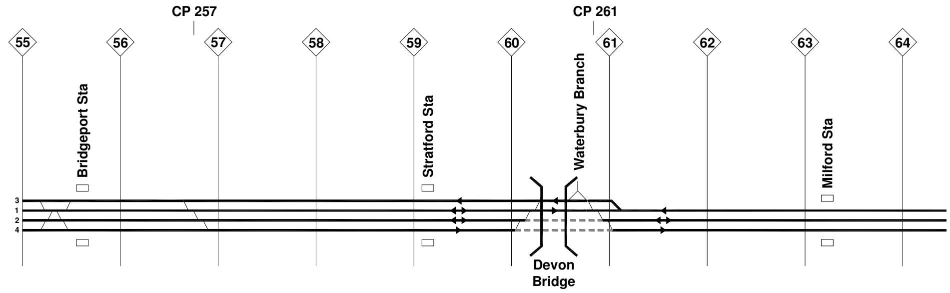


Figure V-4: Staging Alternative A – Stage 2

3. Staging Alternative B

This alternative also includes two stages, but utilizes the existing railroad plant to accomplish the staging. Similar to Alternative A, this alternative requires nine (9) miles of two track service during Stage 1, and one (1) mile of two track service during Stage 2. However, this scenario will affect operational flexibility due to the method by which the Waterbury Branch is routed to Track 1 during construction of the northerly half of the bridge.

Stage 1

Stage 1 would see a long-term track outage between CP 257 and CP 261 at Track 3 and between CP 257 and CP 266 at Track 1. Westbound trains would be switched to Track 2 at CP 266 west of the Milford Station, and remain on Track 2 until CP 257 west of the Stratford Station. Eastbound trains would switch to Track 4 at CP 257. Trains from the Waterbury Branch headed westbound on the New Haven Line would be required to first travel east to clear the interlocking at CP 261 to switch to Track 2, and then reverse direction and continue west. The Stratford and Milford Stations would be affected during this substage. Bridge plates across Tracks 1 and 3, or a temporary platform would be required at the Stratford Station to facilitate inbound passengers boarding the trains on Track 2. Similarly, bridge plates across Track 1 would be required at the Milford Station to facilitate inbound passengers. During the Stage 1 permanent track outage, the northerly half of the bridge would be rehabilitated or replaced.

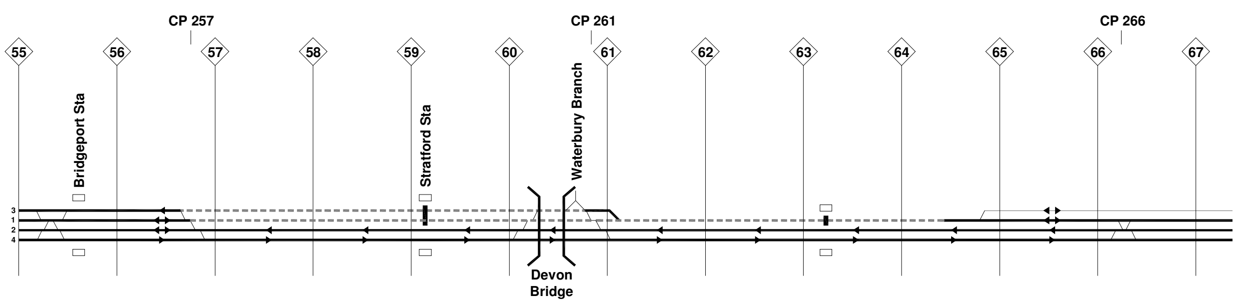


Figure V-5: Staging Alternative B – Stage 1

Stage 1 – Alternative

The nine (9) mile two track service and bridge plate requirements of this stage could be eliminated by adding a "cut and throw" between Track 1 and 2 between the east end of the bridge and the interlock at CP 261 similar to the one required for Alternative A. This would reduce the two track service to four (4) miles instead of nine (9) miles, and would eliminate impacts to the Milford Station.

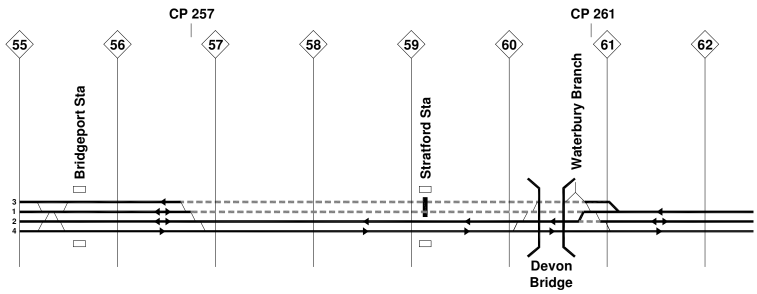


Figure V-6: Staging Alternative B – Stage 1 Alt

Stage 2

Stage 2 for this alternative would be identical to Stage 2 of Alternative A, and would involve a long-term track outage at CP 261 of Tracks 2 and 4 to facilitate construction relating to the southerly half of the bridge. With this configuration in place, westbound trains would be switched to Track 3 at MP 61 east of the bridge, and either remain on Track 3 or return to Track 1 at the interlocking west of the bridge. Eastbound trains would be shifted to Track 1 at the CP 261 interlocking just west of the bridge, and would return to either Track 2 or Track 4 at the interlocking east of the bridge. Trains from the Waterbury Branch would maintain their original routing to and from Track 3. During the Stage 2 long-term track outage, the southerly half of the bridge would be rehabilitated or replaced. No stations would be affected during this stage.

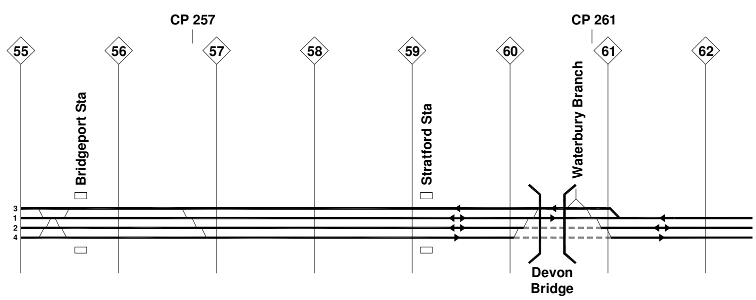


Figure V-7: Staging Alternative B – Stage 2

4. Other Considerations

Any significant reconstruction work will result in impacts to operations between CP 255 and CP 261 at a minimum. A minimum of four (4) miles of two track service is necessary unless a new series of interlockings are added between CP 255 and CP 261. This two track service will affect the operational flexibility of the railroads. By only allowing for one track in each direction, peak hour service, especially as relates to express and local trains, would be adversely affected.

VI. Marine Operations and Navigational Requirements

Marine operations in the Housatonic River consist of primarily small pleasure craft, fishing vessels, local law enforcement vessels, US Coast Guard vessels, and channel maintenance vessels (for dredging). The bridge is depicted on Nautical Chart No. 12370 "North Shore of Long Island Sound Housatonic River and Milford Harbor", published by the National Oceanic and Atmospheric Administration's Office of Coast Survey. A portion of this chart is shown in Figure VI-1. The navigation channel at the Devon Bridge is listed with a horizontal clearance of 83 feet, a vertical clearance of 19 feet (bridge in closed position), and a vertical clearance of 65 feet (bridge in open position, controlled by Moses Wheeler Bridge).

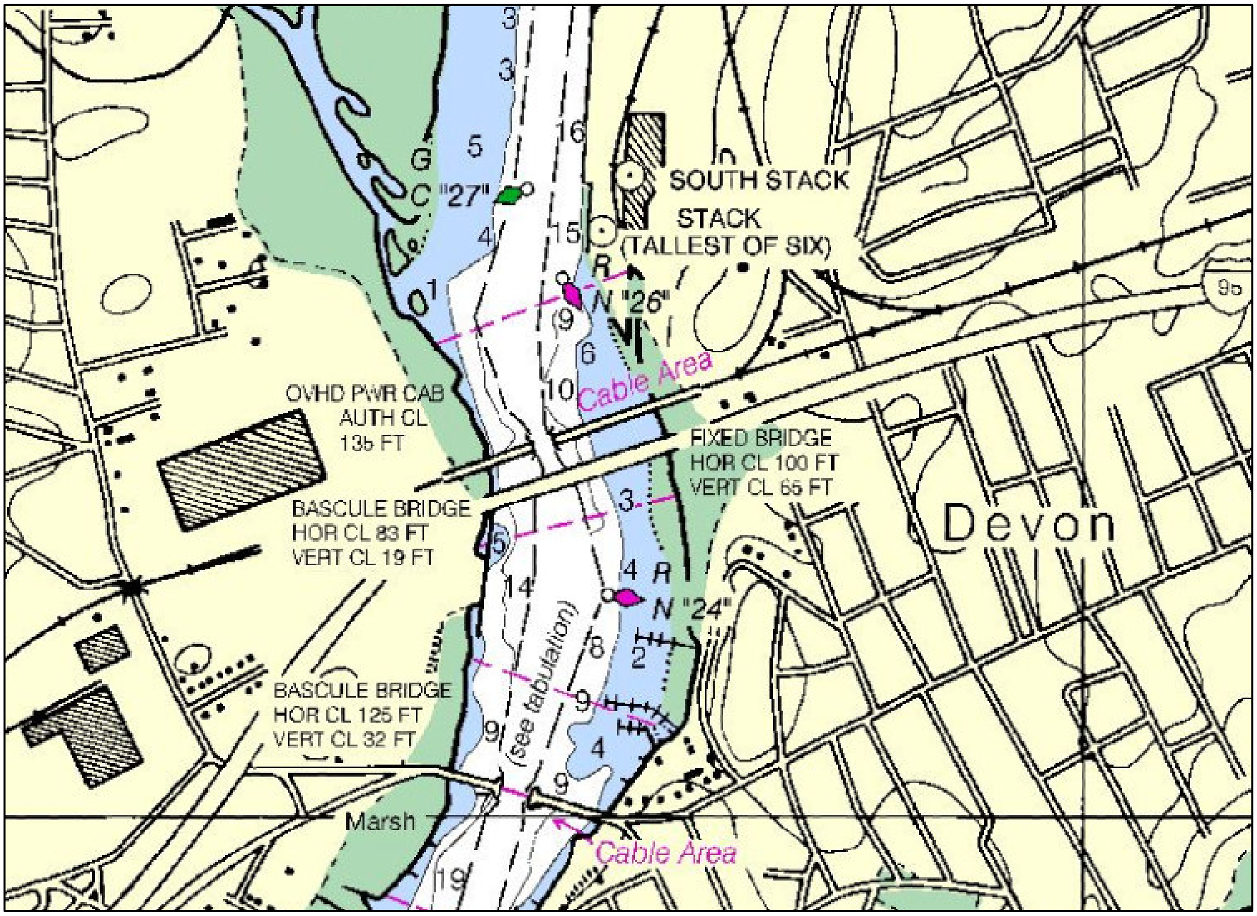


Figure VI-1: NOAA Chart No. 12370 "North Shore of Long Island Sound Housatonic River and Milford Harbor"

Stantec reviewed the bridge opening logs for the period between 2006 and mid-2009. A review of the bridge opening logs indicates that the bridge is opened approximately 95 times per year, with an average of 85 openings for vessels. The remainder of the openings were performed for testing bridge operations. Openings for vessels were primarily in the months of April through November. Charts depicting bridge opening events by month for the calendar years of 2007 and 2008 are shown in Figures VI-2 and VI-3 respectively.

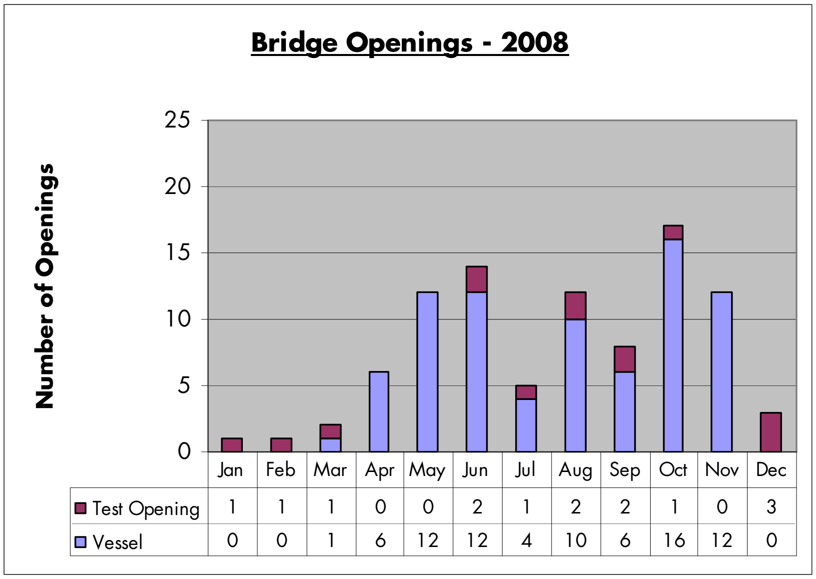


Figure VI-2: Bridge Openings – 2008

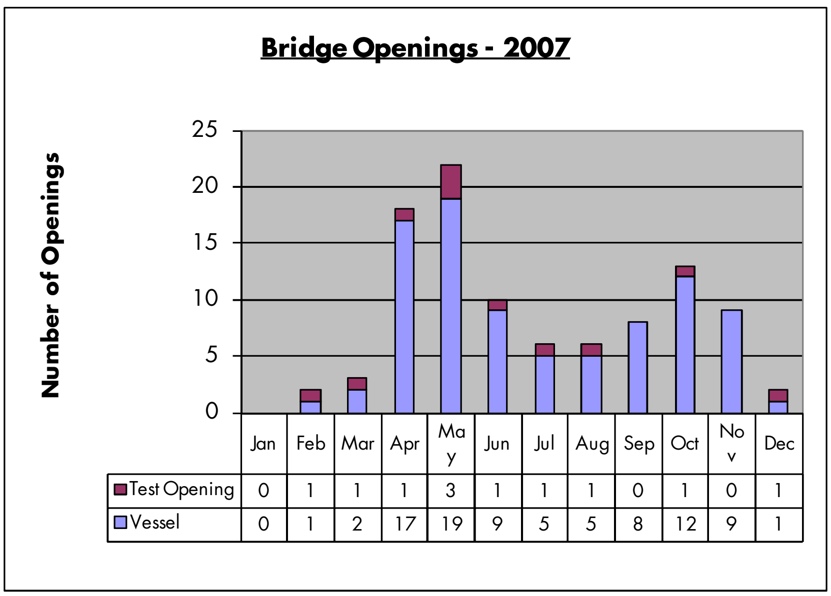


Figure VI-3: Bridge Openings – 2007

It is anticipated that the Coast Guard will require that any rehabilitation or replacement of the existing bridge provide the existing horizontal and vertical clearances at a minimum. The proposed Moses Wheeler Bridge to the south will provide 70.9 feet of vertical clearance above mean high water; it is anticipated that at least this clearance will be required for any replacement Devon Bridge structure. Similarly, any replacement structure will likely need to provide 100 feet of horizontal clearance in the navigation channel to match the proposed Moses Wheeler Bridge.

A meeting with the Coast Guard will occur subsequent to the submission of this report.

A survey consisting of field and telephone interviews was conducted with current and possible future users of the navigable portion of the Housatonic River. The results are tabulated below:

Caswell Cove Marina

Contact: Dave Phillips, Manager

Phone: (203) 876-9880

Location: Immediately upstream of bridge

Comments: The marina houses approximately 110 vessels, with lengths up to 80 feet. Approximately 3 or 4 vessels require opening of the bridge to access points downstream of the bridge. These boats are generally operated in the months of April through October, and are used a couple of times per week. Mr. Phillips noted that the bridge presents an inconvenience to boaters, and keeps people from using the river. He noted that the movable span is inoperable at times, preventing access under the bridge for large boats. He also noted that boaters need to make an appointment for the bridge to open.

Enterprise Yacht Sales / Beacon Point Marine

Contact: Rick Kral, Owner

Phone: (203) 929-7444

Location: Upstream of Sikorsky Aircraft

Comments: The marina houses roughly 200 vessels, one-third of which are sailboats that require opening of the bridge to navigate to downstream. He noted that scheduling an opening can be very difficult, and that often boaters will coordinate their trips with other boaters to minimize the use of the movable bridge.

Ayer's Landing Marina

Contact:

Phone: (203) 924-4023

Location: Upstream of Sikorsky Aircraft

Comments: Was unable to contact at this time.

River's End Marina

Contact: Bernie Shaw, President

Phone: (203) 924-4167

Location: Adjacent to Ayer's Landing

Comments: The marina houses approximately 80 vessels up to 45 feet in length during the warmer months, but has storage capabilities for up to 140 vessels. Mr Shaw noted that they house 8 sailboats that may require use of the bridge to navigate under the bridge.

Sikorsky Aircraft

Contact: Chief Anthony Dynderski

Phone: (203) 386-6688

Location: Upstream of bridge

Comments: Sikorsky has two boat launches that it uses for fire and rescue operations. Their largest vessel is 30 feet in length.

NRG Energy

Contact:	Steve ?
Phone:	(203) 783-6213
Location:	Immediately Upstream of bridge
Comments:	Was unable to contact at this time. However, a review of meeting minutes from the Connecticut Maritime Commission indicates that NRG had historically used the bridge to provide fuel for steam turbines but has since decommissioned the turbines. The minutes also reflect that NRG uses the bridge for large construction deliveries (turbines, etc.) when the facility is being worked on.

VII. Scour Assessment and Hydraulics

The information in this section is compiled from available data produced for the replacement of the Moses Wheeler Bridge No. 00135, the 2001 In-Depth Inspection Report for Bridge No. 08080R, prepared by McLaren/DiCesare Consulting Engineers, and the current In-Depth Inspection conducted as part of this project.

A.) Existing Hydraulic Conditions

The Metro North Railroad (MNRR) Devon Bridge is parallel to and 43 feet north of the proposed Moses Wheeler Bridge carrying I-95. The Scour Assessment and Hydraulics for the replacement of the Moses Wheeler Bridge included an analysis on the existing, proposed and temporary impact to the Devon Bridge.

1. Existing Devon Bridge

The Devon Bridge was originally built in 1905 and reconstructed in 1991. It spans the Housatonic River providing a navigation horizontal and vertical clearance of 83 and 19 feet respectively.

There are limited construction plans available for the bridge due to its age. However, based on underwater inspections performed in 1988 by Boswell Underwater Engineering, the CTDOT contracted Non-Destructive Testing (NDT) methods to determine the elevations of the footings for Piers 2 and 3. A 1906 plan entitled "Plan of New and Old Bridges and Channels", found in the CTDOT map room, detailed drawings of the foundations for each of the existing piers. Although not documented as an "As-Built", the dimensions shown closely correlate with those determined during the NDT of Piers 2 and 3. This 1906 plan was used to give dimensions of the foundation for the scour assessment in this report.

2. Housatonic River

The Devon Bridge crosses the Housatonic River between the Town of Stratford in Fairfield County and the City of Milford in New Haven County. The river and its tributaries drain an area of 1,948 square miles and classified at Watershed Basin No. 6 by the Connecticut Department of Environmental Protection (CTDEP). It enters Long Island Sound at Milford Point approximately three miles downstream. Within the Housatonic River watershed are several lakes formed by hydroelectric power dams. When normal river flows are inadequate, there is a periodic constriction of flow behind the dams; however, none of the impoundments were developed for flood control.

3. Hydrology

Hydrologic boundary conditions were developed for the replacement of the Moses Wheeler Bridge. The original study method for the 1,948 square mile watershed included evaluation of United States Geological Survey (USGS) stream gauge data at the Stevenson Dam in Monroe, CT and on the Naugatuck River in Beacon Falls. In accordance with the Connecticut Department of Transportation Drainage Manual (CTDOTDM), gauged data cam be transferred up or downstream on a gauged stream if the drainage basin for the study location is ≥ 75% or ≤ 125% of the drainage basin at the gauge with application of the transfer equation.

Subsequent to the original analysis, the USGS released a report "Peak Discharges for Streams in Connecticut for Selected Recurrence Intervals". Revised stream flow statistics, analyzed from historical gauge data, and transferred to the basin area at the bridge, generated the following design flows used the hydraulic and scour analysis of the Moses Wheeler Bridge.

Design Flows	
Return Period	Discharge (cfs)
2 – Year	27,831
10 – Year	65,903
25 – Year	94,932
50 – Year	122,548
100 – Year	156,685
500 – Year	274,408

4. Channel Conditions

Underlying conditions of the Housatonic River are mapped as Derby Schist bedrock underlying surficial deposits. Investigation of subsurface strata indicated 10 to 36 feet of organic silt/sands within the banks of the river. Sand/gravel and glacial till underlie the organic layer. Schist bedrock was approximately 6 feet to 98 feet deep from the land to the river respectively.

The Housatonic River is a wide perennial watercourse with moderate valley relief. Its channel boundaries are semi-alluvial with a flood plain that is absent or less than two times the width of the river. There is some local river branching and braiding but the river is generally straight with random variations in width and development of bars.

5. Waterway/Tidal Characteristics

The Housatonic River is tidally influenced for the 13 miles between the Long Island Sound and the Derby/Shelton Dam. Tide data from the "Tidal Flood Profiles, New England Coastline" between the Stratford/Milford Point are as follows.

Tide Data	
Mean High Water	2.91 (NAVD)
Mean Low Water	-3.84 (NAVD)
Tidal Range	6.75 feet
Tidal Period	12.5 hours

Pertinent information on the tidal effect, with regard to the hydraulics of the bridge, is provided in the following section.

B.) Fathometric Survey

A fathometric survey was conducted in conjunction with the underwater inspection of the bridge. This survey encompassed the channel up- and downstream of the bridge. See Figure G-4. Although no undermining was noted, previous underwater inspection reports, along with the current inspection results, indicate active scour (aggregation and degradation) is occurring at all piers.

C.) Assessment of Scour Potential

The hydraulic analysis for the Moses Wheeler Bridge involved three steps in order to determine the tidal conditions for the design storm. The first step being a steady state analysis with design discharges for the upstream boundary conditions and a known water surface elevation from the normal tide hydrograph for downstream conditions. The second step used the resulting steady state models to run a transient analysis where the downstream conditions were a constantly changing stage elevation to model the tidal properties of Long Island Sound.

In determining the applicable upstream and downstream boundary conditions, consideration of the watershed size and time of concentration for the inland flow resulted with the following combinations of storm surges and discharge events.

Hydraulic Scenarios Applied for Hydraulic Study		
Event	Upstream Boundary Condition	Downstream Boundary Condition
100-Year Storm Surge	Average Daily Flow	100-Year Surge with High Tide
500-Year Storm Surge	Average Daily Flow	500-Year Surge with High Tide
100-Year Flood	100-Year Discharge	Normal Tide
500-Year Flood	500-Year Discharge	Normal Tide

These scenarios were acceptable because the tidal surge would likely recede in advance of the inland flow wave from the Housatonic River Watershed. From the successful results of the unsteady analysis, flow and stage hydrographs were extracted from the model at the cross sections used as boundary conditions for the third step that created the two dimensional finite element network used for the in-depth analysis. These two sections were located 1,740 feet north of the Devon Bridge and 755 feet south of the Moses Wheeler Bridge.

The results of the two-dimensional hydrodynamic modeling for each of the hydraulic scenarios previously noted are as follows:

Peak Discharge and Water Surface Elevations for Devon Bridge for proposed conditions of the Moses Wheeler Bridge				
Event	Peak Discharge* (cfs)		Peak Water Surface Elevation* (NAVD 1988)	
	Flood Direction	Ebb Direction	Flood Direction	Ebb Direction
100-Year Tidal Surge	48,134	53,466	9.5	9.2
100-Year Riverine Flood		176,219		16.4
500-Year Tidal Surge	55,585	65,579	10.5	10.5
500-Year Riverine Flood		270,968		16.4

*Note – Peak discharges and peak water surface elevations may not occur under the same time step.

Due to the close proximity of the Devon Bridge, the analysis evaluated the effects of scour on the Devon Bridge during the entire construction phase of the Moses Wheeler Bridge and the anticipated scour affects on the Devon Bridge after the Moses Wheeler Bridge is replaced.

1. Long Term Scour

Several resources and a 1994 Scour Evaluation Report by Greiner documented that there is negligible evidence of long term bed instability. Since the navigational channel is maintained by the Army Corp of Engineers for a constant streambed elevation, the vertical stability is preserved. The final assessment was that there is no long-term degradation or aggregation of the streambed or noticeable lateral migration.

2. Contraction Scour

This component of scour results from a contraction of the flow area at the bridge. This causes an increase in velocity and shear stress on the natural channel bed at the bridge.

For each of the events being analyzed, the Devon Bridge experienced contraction scour. The Moses Wheeler Bridge only experienced contraction scour during the temporary conditions analysis.

In order to study the contraction scour depth on the Devon Bridge, the greatest equated velocities at the bridge were selected. Created observation points, up and downstream of each pier nose, provided the information needed to graph the average time of maximum velocity. Once this was established, flow rate was determined at the applicable sections. The following table documents the scour conditions pre and post-replacement of the Moses Wheeler Bridge.

Existing and Proposed Contraction Scour Depth (feet)				
Event	Existing Conditions (feet)		Post-Replacement Conditions (feet)	
	Flood Direction	Ebb Direction	Flood Direction	Ebb Direction
100-Year Tidal Interval	1.93	2.00	2.40	2.43
100-Year Riverine Interval	4.23		3.54	
500-Year Tidal Interval	2.33	2.36	2.82	2.53
500-Year Riverine Interval	4.82		4.26	

3. Local Scour

Local scour occurs when material is removed from around piers, abutments, and embankments caused by an acceleration of stream flow and resulting vortices.

The analysis is considered to be conservative for the following reasons: tidal environment in this segment of the river and lack of research on time dependent local scour computations. The HEC-18 manual methods were used to determine abutment and pier scour. The time step used in the contraction scour analysis was used as the scour critical time step for the local scour analysis.

4. Pier Scour

The Devon Bridge is supported on stone masonry piers with a sharp nose facing upstream into the flow of the river, and a blunt or square nose facing downstream. The piers are supported on deep concrete foundations with the same nose configurations as the pier it supports, excluding piers four and six, which sit on rectangular foundations.

The Devon Bridge elevations of the river bed at the faces of piers (based on May 2009 Fathometric Survey), and bottom of foundations used for the Moses Wheeler Bridge scour analysis are provided in the following table.

Devon MNRR Bridge River Bed and Foundation Elevations (NAVD88)						
Bed Elevation	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6
Upstream	-11.0	-18.0	-20.5	-12.8	-9.5	-3.5
Downstream	-10.5	-19.1	-20.0	-24.0	-8.0	-1.5
Foundation Elevation*	-30.84	-42.98 (-40.0)	-43.63 (-40.0)	-57.41 (-44.0)	-29.85 (-44.0)	-45.93

*Estimated on limited foundation information
(XX.X) indicates elevations calculated based on ultrasonic testing

Worthy of note is that the previously assumed bottom of footing elevations for Piers 4 and 5 vary significantly from those determined in December 2004 by NDT Corporation, which indicate bottom of footing elevations for these two piers at EL -44.0. Results from the ultrasonic testing are used where available when determining impacts from scour.

Although the proposed pier geometry of the Moses Wheeler Bridge results in a decrease in local scour, the consequence is an increase in local scour on the Devon Bridge. The majority of local scour increases are less than 1.64 feet with the greatest increase at pier two during the 100-year storm surge on the ebb phase of the tide cycle. This difference is 5.45 feet.

Additional analysis examined undermining of the Devon Bridge piers for all hydraulic scenarios incorporating contraction scour, local scour, bed elevation, and elevation of the bottom of the footing/pile cap. Contraction and local pier scour were added in to determine the susceptible piers. The results are summarized on the following table.

Existing and Proposed Hydrologic Scenario Scour Depths						
Event	**Devon Bridge**					
Existing Conditions	**Pier 1**	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	**Pier 6**
100-Year Tidal Interval						
Flood Direction	***31.89***	***32.64***	***31.30***	29.85	***29.66***	18.73
Ebb Direction	17.71	21.85	21.19	26.02	***28.48***	16.83
100-Year Riverine Interval	***27.07***	***34.97***	***32.61***	36.38	***41.40***	31.59
500-Year Tidal Interval						
Flood Direction	***32.91***	***34.42***	***33.10***	31.63	***32.15***	19.91
Ebb Direction	19.03	23.29	***22.54***	26.44	***28.38***	17.16
500-Year Riverine Interval	***31.82***	***36.58***	***35.46***	***39.60***	***45.08***	38.22
Proposed Conditions	**Pier 1**	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	**Pier 6**
100-Year Tidal Interval						
Flood Direction	***34.74***	***35.83***	***34.51***	32.71	***30.61***	18.77
Ebb Direction	<u>***22.08***</u>	<u>***27.72***</u>	<u>***25.62***</u>	26.67	***29.20***	17.35
100-Year Riverine Interval	***28.57***	***34.78***	***32.41***	36.25	***41.27***	31.40
500-Year Tidal Interval						
Flood Direction	***34.02***	***34.78***	***33.56***	32.84	***33.04***	21.29
Ebb Direction	19.49	23.72	***22.96***	26.93	***30.22***	19.29
500-Year Riverine Interval	***31.40***	***36.19***	***35.07***	***39.17***	***46.52***	37.86

Bold Italicized indicates pier undermined during event.

Notably the piers being undermined during existing conditions will continue to be undermined during proposed conditions with the addition of the undermining of pier 1, 2 and 3 (underlined) during the 100-year tidal interval in the ebb direction. Note that the 500-year ebb tide flow does not undermine piers 1 and 2 where the 100-year ebb tide flow does. This is attributed to the scour computation that is sensitive to the angle of attack and the 100-year scenario has a greater angle resulting in greater scour depths.

5. Abutment Scour

The Devon Bridge abutments are gravity type with wing walls considered to be founded on spread footings. Both abutments retain the approach embankments. The east abutment is not exposed to flow during any of the hydrologic conditions. During the extreme hydrologic events, the west abutment does interact with the flow. A storm surge however only interacts with the west abutment for a short period and does not warrant concern so the analysis evaluated the riverine flow only. Those computed abutment scour depths are as follows.

Computed West Abutment Scour Depths		
Hydrologic Event	Scour Depth (Feet)	
	Existing Site Conditions	Proposed Site Conditions
100-year Discharge	10.00	12.07
500-year Discharge	15.52	16.57

It was determined that these computed scour depths for the west abutment would not be achieved for the following reasons:

- The plans found for the Devon bridge substructure show that the abutment is founded on rock.
- Large diameter riprap line the bank at and around the west abutment
- No previous evidence of scour has been noted.

D.) Summary

The existing Devon Bridge appears to be hydraulically adequate, with adequate freeboard between the low chord and highest calculated flood elevations. The scour depths calculated for the Devon Bridge are slightly affected in the analysis of the future Moses Wheeler Bridge; however, due to the tidal nature of this reach in the Housatonic River and storm surge period, maximum scour depths would not be reached. Additionally, the Devon Bridge has been in service for more than 100 years and subject to several experiences exceeding the 100-year storm event.

A detailed hydraulic and scour analysis of the bridge will be required to confirm flood elevations and predicted scour at the Devon Bridge, and will be based on the layout of the proposed structure. Until that analysis has been completed, and for the purposes of this report, it is assumed that the low chord of any new structure should be at or above the existing low chord elevation.

Proposed work at the bridge with respect to scour will depend on the chosen rehabilitation alternative.

Recommendations for any alternative which retains the existing substructure units include:

- Perform NDT on Piers 1 and 6 to determine foundation depths;
- If a detailed scour analysis indicates the one or more of the piers are scour susceptible, install an automated scour monitoring plan on Pier 3 and Pier 5 in accordance to HEC-23 “Bridge Scour and Stream Instability Countermeasures”; and
- After meeting all environmental concerns, demolished Moses Wheeler Bridge piers could be used as riprap around the Devon Bridge piers. Further investigation as to the durability of the concrete as a countermeasure is warranted before considering this a viable alternative.

Any new substructure units that are to be constructed will require deep foundations founded below calculated scour depths.

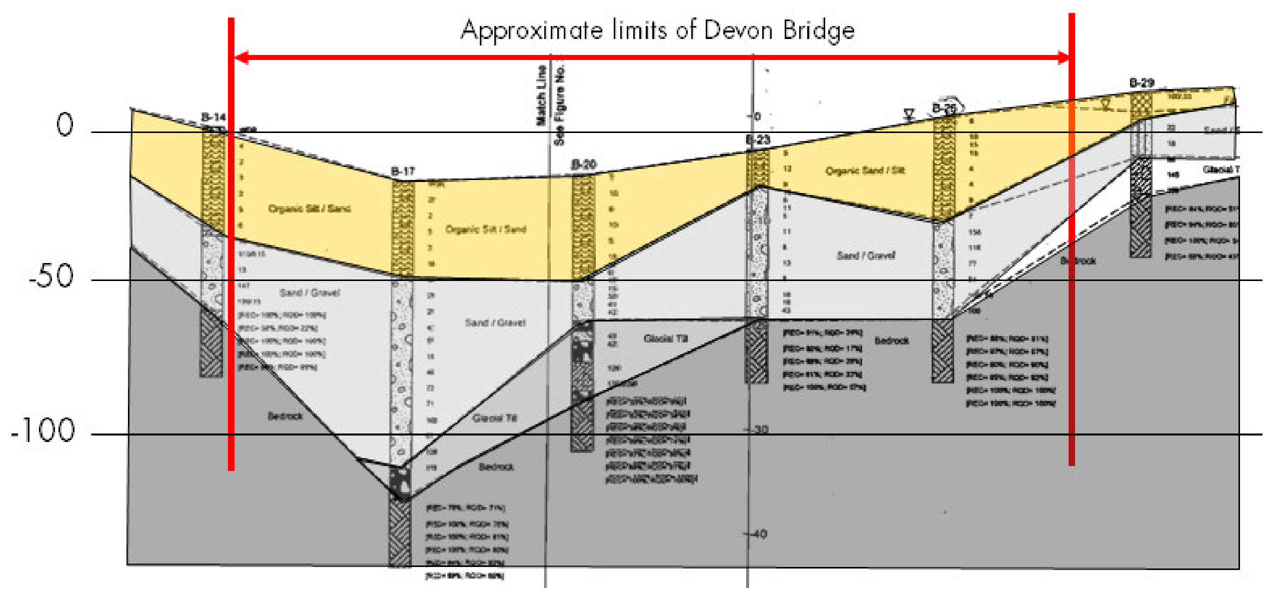
VIII. Soils and Foundations Assessment

A significant subsurface exploration program was conducted to support the design efforts for the replacement of the Moses Wheeler Bridge (Bridge No. 00135). The results of this program, along with interpretation of the results, is included in the report *Structure Soils and Foundation Report Bridge No. 135*, July 2002, prepared by GeoDesign Incorporated. The Moses Wheeler Bridge runs parallel with the Devon Bridge, and is located approximately 90 feet to the south. For the purposes of this feasibility report, and as directed by CTDOT, the findings presented in the Moses Wheeler Bridge foundation report are assumed to be applicable to the Devon Bridge.

A total of fifty four (54) soil borings were completed during the design phase of the Moses Wheeler Bridge replacement. These borings supplemented the eighty (80) borings completed and presented on the contract drawings for the original highway bridge. Of these borings, approximately twenty-one (21) are in the vicinity of the Devon Bridge. These borings are located at the approximate locations of the proposed Moses Wheeler Bridge piers, and include one rock core at each of the proposed piers. These piers are identified in the GeoDesign report as Piers 5, 6, 7, 8, 9 and 10.

Within the Devon Bridge study area, the GeoDesign report indicates that subsurface conditions consist of a top layer of generally loose organic silt/sand approximately 30 feet thick. Below this layer is a layer of medium to dense sand/gravel of varying thickness. At several locations near the center of the river, glacial till is present. Schist bedrock was encountered at depths between 60 and 100 feet. Bedrock was cored to a depth of 10 feet at each proposed pier location, revealing a somewhat variable Recovery and Rock Quality Density (RQD). In general, RQD values increased proportionate to the depth of the core. The GeoDesign report indicated that, within the sand/gravel layer, cobbles and boulders were encountered frequently during the boring program.

The information in this section is compiled from available data produced for the replacement of the Moses Wheeler Bridge No. 00135 and a 2001 In-Depth Inspection Report for Bridge No. 08080R, prepared by McLaren/DiCesare Consulting Engineers.



A.) Existing Substructure System

The existing bridge is supported by two gravity type masonry abutments founded on spread footings, with six intermediate piers constructed of masonry with deep concrete foundations founded on spread footings. The abutments are labeled Abutments 1 and 2, from west to east. The piers are labeled Piers 1 through 6, again from west to east.

Photo VIII-1: Typical Pier

A timber pile supported fender system protects Piers 3 and 4, which are adjacent to the navigation channel below movable Span 4.

Photo VIII-2: Fender System

Two investigations were conducted prior to this project to determine the bearing elevation (bottom of footing) of Piers 2 through 5. Piers 2 and 3 were investigated in October 1999 by Clough Harbor & Associates under State Project 300-0033, and utilized non-destructive wave dispersive wave testing to obtain data. Piers 4 and 5 were investigated in December 2004 by NDT Corporation under Purchase Order number 500118842, and utilized a similar non-destructive sonic/ultrasonic reflection methodology to obtain data.

These reports indicate that the bottom of footing elevations are as follows:

Bottom of Footing Elevations	
Substructure Unit	Elevation
Abutment 1	unknown
Pier 1	unknown
Pier 2	-40.0
Pier 3	-40.0
Pier 4	-44.0
Pier 5	-44.0
Pier 6	unknown
Abutment 2	unknown

Based on the boring information provided in the GeoDesign report, these footings appear to bear upon the dense sand/gravel layer, and do not bear on bedrock. Both reports also indicated that no evidence of piles was noted during the ultrasonic testing.

B.) Proposed Substructure System

The proposed substructure system depends on the final rehabilitation/replacement alternative selected.

1. Short Term Repairs

The short term repair alternative incorporates the existing substructures with no substantial changes.

2. Rehabilitation

The rehabilitation alternative incorporates the existing substructures. Unlike the short term repair alternative, improvements to the piers will be made to improve seismic performance due to the design service life horizon. The existing masonry piers will be strengthened by one of the following:

- Wrapping the piers with fiber or steel mesh encasement;
- Encasing the piers in reinforced concrete; or
- Post-tensioning the piers (horizontally for confinement) using external steel tensioning rods.

Applying one of the above retrofit procedures will improve the seismic resistance of the piers by increasing their ductility and thus the loads applied to each pier. These retrofits will need to be completed both above and below the waterline, complicating the repair procedures.

There is some risk inherent in performing repairs to the piers, in that the internal composition of the piers is not entirely known. Thus, there is the risk that the piers may be damaged during retrofit procedures such as drilling and grouting dowels into the masonry. In addition, the piers will need to be dewatered to perform such repairs effectively, requiring the installation of a cofferdam system. This may prove especially difficult for alternatives where the superstructure are to remain in place (Alternatives II & IIIa).

3. Superstructure Replacement

The substructure repair/retrofit procedures performed with this alternative will be similar to those performed as the Rehabilitation alternative, with the addition of the modification of the pier and abutment bridge seats to accept a new superstructure.

4. Full Replacement

The GeoDesign geotechnical report for the replacement of the Moses Wheeler Bridge recommends a deep foundation system consisting of drilled shafts socketed into bedrock. The subsurface conditions likely do not significantly vary 100 feet to the north of I-95, and given that the loadings on the foundation will be of a similar order of magnitude, a similar deep foundation system would be appropriate for the Devon Bridge.

Thus, the substructure system necessary for an entirely new structure would likely be similar to the foundation recommended for the Moses Wheeler Bridge. This would consist of a deep foundation system of drilled shafts which extend above the waterline to a horizontal pier cap/bridge seat tying the shafts together. Shafts will use permanent casings seated at the top of the bedrock.

The use of drilled shafts allows for minimal use of cofferdam and dewatering, as they can be installed from the barges and dewatered using the permanent casings. Other deep foundation types, such as steel H-piles, will require a cofferdam with dewatering around each of the proposed piers. The cofferdam installation would introduce additional cost and complication due to installation headroom clearance requirements between the construction stages. In addition, some excavation of the in situ stream bed material would be required, which would increase the risk of encountering contaminated soils and water.

If concerns of the aesthetics of the new drilled shafts becomes a concern, the columns could be faced with a stone veneer similar what is being provided for at the Moses Wheeler Bridge. Alternatively, the bottom of the pier cap could be extended to several feet below the low tide line, creating the effect of a solid wall pier. This wall could be faced with brownstone to match the appearance of the existing piers. Stones from the existing piers could be used as a facing.

IX. Environmental and Permitting Concerns

A.) General

Six alternatives have been developed for the repair/rehabilitation/replacement of the Devon Bridge, each of which has different environmental and/or permitting requirements.

Alternative I involves repairing the existing bridge structure from a barge in the Housatonic River. All of the repairs will be to the superstructure (the part of the bridge above water).

Alternative II, like Alternative I, involves repairing the existing bridge structure from a barge in the Housatonic River. The difference between Alternative I and Alternative II is that the scope of the repairs in Alternative II is more extensive, with work to the superstructure and the substructure (the part of the bridge below the water). For example, Alternative II will involve wrapping the piers with fiber or steel mesh encasement, encasing the piers in reinforced concrete, or post-tensioning the piers (horizontally for confinement) using external steel tensioning rods. The masonry substructure will be repointed (below water) where existing mortar has failed or exhibits cracking and piers will be retrofitted to improve seismic performance. It is possible that a platform, with pilings in the water, will be constructed along the riverbank to facilitate this repair work.

Alternatives IIIa and IIIb both involve repairing the existing bridge structure from a barge in the Housatonic River. The scope of the repairs in Alternative IIIa and IIIb is more extensive than Alternative I, with work to the superstructure and the substructure (the part of the bridge below the water). The scope of the repairs for Alternatives IIIa and IIIb is also more extensive than for Alternatives II, involving modification of the pier and abutment bridge seats to accept a new superstructure. It is possible that a platform, with pilings in the water, will be constructed along the riverbank to facilitate this repair work.

The main difference between Alternative IIIa and IIIb is that in Alternative IIIa, the superstructure will be partially replaced, while Alternative III calls for the complete replacement of the superstructure.

Alternatives IVa and IVb involve a complete replacement of the entire bridge structure, which will require demolition of the existing structure and digging a deep foundation system for the new bridge in the same location where it currently exists. The foundation system will involve drilling shafts socketed into bedrock. Some of this foundation work will take place under water. The foundation system would begin below and extend above the waterline. While much of the construction work will take place from a barge in the Housatonic River, it is likely that a platform, with pilings in the water, will be constructed along the riverbank to facilitate construction. Dewatering will be required for drilling shafts and deep foundation work. In addition, some excavation will be required in the river bed, increasing the potential for encountering contaminated soil and water.

B.) Regulatory Framework and Resources

In order to establish the regulatory framework within the project area, resources that are regulated need to be identified. These resources determine what types and levels of permits may be necessary, and from which regulatory agency. The pertinent setting for the project and permits triggered is provided as follows.

1. NAVIGABLE TIDAL WATERWAY

All six alternatives would take place within navigable waters of the United States. The bridge spans the Housatonic River, a navigable water and a tidal river. The elevation of the ordinary high water line (OHW) at the bridge site will need to be identified as the regulatory limit for the purposes of permits authorized by the U.S. Army Corps of Engineers (USACE), as described below. The elevations of the high tide line (HTL), mean high water (MHW), and mean low water (MLW) at the bridge site will need to be identified as the regulatory limit for the purposes of permits authorized by the Connecticut Department of Environmental Protection (CTDEP), as described below. The construction activities below or within these regulated areas will be subject to permits.

The suite of permits applicable to the Devon Bridge project on account of its location across navigable and tidal waterways are described below:

- U.S. Coast Guard Bridge Permit: A U.S. Coast Guard bridge permit is required for projects that modify bridges across a navigable water. A permit is not required for routine maintenance or replacement of worn or obsolete bridge parts, but is required if a bridge will be replaced and if replacing any of the bridge parts will alter the structural configuration or navigational clearances, significantly modify any substructure or superstructure components, or violate any navigational conditions of the original permit.

  Alternatives I and II will likely not require a U.S. Coast Guard Bridge Permit, as they involve maintenance and replacement of existing bridge parts and will not change navigational conditions. Alternative IIIa may require a U.S. Coast Guard Bridge Permit, as the work entails a partial replacement of the superstructure, which may alter the bridge's structural configuration. Further coordination with the U.S. Coast Guard is required to determine if a permit is required for Alternative IIIa. A U.S. Coast Guard Bridge permit will be required for Alternative IIIb, which involves complete replacement of the bridge's superstructure, and Alternative IVa and IVb, which involve a full bridge replacement.

  CTDOT anticipates obtaining this permit through coordination with the First Coast Guard District in New York. The Coast Guard issues its permit only after all other approvals are obtained.

- U.S. Army Corps of Engineers Programmatic General Permit (PGP) Category II or Individual Permit: The USACE regulates construction and other work in navigable waterways under Section 10 of the Rivers and Harbors Act of 1899 and the discharge of dredged or fill material into "waters of the United States" under Section 404 of the Clean Water Act. "Waters of the United States" are navigable waters, tributaries to navigable waters, wetlands adjacent to those waters and/or isolated wetlands that have a demonstrated interstate commerce connection. The Housatonic River is a navigable waterway.

  An application to the USACE New England Division will be required for a Section 10 and 404 permit for any of the alternatives. Depending on final design impacts, a PGP with a Connecticut Addendum will be sought for Alternatives II, IIIa, and IIIb, if, as anticipated, they do not impact tidal wetlands. For example, if a platform is needed along the riverbank to facilitate construction, it may (or may not) impact tidal wetlands. An individual permit will be sought for Alternative IVa and

IVb, if, as anticipated, it does result in impacts to tidal wetlands (i.e., from digging for the foundation system).

- CTDEP Structures, Dredging and Fill Permit: The CTDEP Office of Long Island Sound Programs will require a Structures, Dredging, and Fill Permit if there will be work waterward of the High Tide Line (HTL). The High Tide Line (HTL) is generally located waterward of the Devon Bridge abutments.

  All of the alternatives except Alternative I are anticipated to involve activities below the HTL and will thus require this permit. The following types of activities below the HTL are noted by the CTDEP regulations:

  - Maintenance or repair of certain existing structures, fill, obstructions, or encroachment
  - The erection of structures including, but not limited to: breakwaters, pilings, booms, culverts, cables, roadways, walkways, and buildings
  - The placement of any obstacle, obstruction or encroachment
  - All work incidental to any of the above activities including: any structure, activity, construction, or site preparation; grading, excavating, dredging, disposing of dredged materials, filling, etc.
  - The removal of vegetation or other material, or other modification of a site, waterward of the HTL

- CTDEP 401 Water Quality Certification: Section 401 Water Quality Certification (WQC) is required when certain federal permits are required (such as USACE permits) associated with potential discharges into wetlands, watercourses, and natural and man-made ponds.

  Since all of the alternatives except Alternative I are anticipated to require a USACE permit – either PGP or individual 404 permit and Section 10 – all of the alternatives except Alternative I are anticipated to need a WQC.

2. COASTAL ZONE BOUNDARY

The Devon Bridge lies entirely within the Connecticut coastal boundary, triggering the need for a CTDEP Coastal Consistency Review for all six alternatives.

- CTDEP Coastal Consistency Review: If new activities or an expansion of existing activities are proposed within the area defined by the Connecticut coastal boundary, then consistency with Connecticut's Coastal Management Act (CCMA) is required.

  All of the alternatives except Alternative I will undergo a Coastal Consistency Review as part of the Structures, Dredging, and Fill permit application. Unless Alternative I will affect regulated resources that fall under the purview of another CTDEP permit (not anticipated at this time), Alternative I will require a stand-alone Coastal Consistency Review.

3. TIDAL WETLANDS

CTDEP Tidal Wetlands Permit: Tidal wetlands are defined by their current or former tidal connection, and their capacity to support certain wetland vegetation. Regulated activity in tidal wetlands includes, but is not limited to:

- Draining, dredging, excavating, or removing of soil, mud, sand, gravel, aggregate of any kind or rubbish from any tidal wetland
- Dumping, filling or depositing upon tidal wetlands any soil, stones, sand, gravel, mud, aggregate of any kind, rubbish or similar material, either directly or otherwise

- Erecting structures, driving piling, or placing obstructions in tidal wetlands

Tidal wetlands are present on both sides of the Housatonic River (see Figure G-1). Alternatives which involve work on the riverbank, such as for excavating and replacing bridge abutments within these wetlands, will impact tidal wetlands. Therefore, it is assumed that Alternative IVa and IVb will require a Tidal Wetlands Permit from CTDEP. Alternatives II, IIIa, and IIIb may also require a Tidal Wetlands Permit, depending on where construction access for these alternatives will be located. Impacts to tidal wetlands will require wetland mitigation at a minimum 2:1 ratio (e.g., 2 acres of tidal wetland replaced for each acre of impacted tidal wetland). Tidal wetlands should be updated after the Moses Wheeler Bridge project is completed, as tidal wetlands will be created along both sides of the river. The presence of such newly formed wetlands need to be incorporated into any construction access plans.

4. INLAND WETLANDS

There may be inland wetlands on the shorelands where work will take place. Connecticut inland wetlands are identified by soil type. A certified soils scientist will need to identify and delineate any inland wetlands that are in the project area. If inland wetlands are identified within the disturbance footprint of any of the alternatives, the potential impacts will need to be evaluated. If impacts cannot be avoided, an Inland Wetlands and Watercourses Permit permit will be required from CTDEP Inland Water Resources Division (IWRD) and, depending on the scale of the impact, a Section 404 permit may be required from the USACE. These permits would be encompassed either through the PGP (noted above in the Navigable Tidal Waterway section) or by applications submitted to both the USACE (for a Section 404 Individual Permit) and to CTDEP IWRD (Inland Wetlands and Watercourses Permit).

5. 100-YEAR FLOODPLAINS

The Devon Bridge is located within Federal Emergency Management Agency (FEMA) mapped 100-year floodplains. Therefore, each of the alternatives was reviewed for its potential to impact the 100-year floodplain, which could trigger the need for a Flood Management Certification administered by the CTDEP.

- CTDEP Flood Management Certification: This permit applies to all State Actions (such as projects funded in whole or in part by the State of Connecticut) in or affecting floodplains or natural or man-made storm drainage facilities. Activities occurring within or affecting a floodplain, or resulting to changes in natural or man-made storm drainage facilities, are subject to Flood Management Certification. Regulated activities include any structure, obstruction or encroachment proposed for emplacement within the floodplain area.

  The FEMA GIS mapping for the Housatonic River at the Devon Bridge shows that the bridge and its environs are located within the 100-year floodplain. All of the alternatives, except Alternative I, require temporary access to the bridge from shore and Alternative IV will additionally entail potential changes to riverbanks. As such, all of the alternatives except Alternative I will likely require Flood Management Certification.

6. THREATENED AND ENDANGERED SPECIES

The Devon Bridge is entirely within an area in which CTDEP has one or more records of threatened and endangered species or critical habitat according to their Natural Diversity Database (NDDB). In order to obtain state and federal permits for the project, the potential presence of state and federally listed species will need to be further investigated. A record does not always mean that a population (species) or their habitat exists there, but further coordination will be required to determine project impacts. All of the alternatives will require this further coordination.

- CTDEP Natural Diversity Database Review: Coordination with CTDEP is necessary to learn more about the potential for threatened and endangered species, their habitats, or other special ecological resources. If CTDEP identifies the presence of and potential impacts to threatened and endangered species from the project, there may be a series of avoidance, monitoring, and/or mitigation measures the project will need to follow. Some of these may be seasonal limitations on construction in the water, if aquatic species are involved.

- Coordination with U.S. Fish and Wildlife Service (USFWS) and National Marine Fisheries Service (NMFS): A written inquiry to the USFWS and NMFS will be required to determine if federally-listed threatened and/or endangered plant or animal species may be potentially impacted by each of the five alternatives and any necessary actions (i.e., seasonal restrictions on construction, avoidance of certain habitat areas, provisions for fish passage) to be taken as a result.

7. OTHER PERMITS

- CTDEP Stormwater and Dewatering Wastewaters from Construction Activities: This General Permit applies to construction activities which result in the disturbance of one or more total acres, which is likely only for the full replacement alternative – Alternative IV.
- CTDEP Registration for the General Permit for Contaminated Soil and/or Sediment Management (Staging and Transfer): Only those alternatives which involve excavation of certain quantities of contaminated soil; Alternatives IVa and IVb could potentially require this permit.

  The Devon Bridge is downstream (approximately 1,000 feet) from a Connecticut Light & Power (CL&P) facility which, according to CTDEP records, has a combined cooling and industrial surface water discharge area and fly-ash lagoon. Due to the proximity of this site, the project area is at risk for hazardous risks. The General Permit for Contaminated Soil and/or Sediment Management authorizes the staging, transfer, and temporary storage of contaminated soil and/or sediment and is intended to address the management of these materials when they are generated during projects that are less than 2 years in duration and involve the excavation of earthen material. The General Permit applies to activities:

    - Greater than or equal to 1,000 cubic yards and less than or equal to 10,000 cubic yards (at any one time) of contaminated soil and/or sediment at the site of excavation for a period exceeding 45 days in duration
    - Transfers, stages, and/or temporarily stores greater than 10 cubic yards and less than or equal to 10,000 cubic yards (at any one time) of contaminated soil and/or sediment at a site other than the site of excavation for any period of time.

8. HISTORIC CONCERNS

The Devon Bridge derives much of its historic significance from its status as one of the original movable railroad bridges on the main line of the New York, New Haven, and Hartford Railroad (NYNH&H RR) in Connecticut, distinctive for turn-of-the-20th century engineering. The Devon Bridge is a Scherzer rolling lift bridge, a type in widespread use during this period, particularly by the NYNH&H RR. Since the bridge was placed on the National Register of Historic Places (NRHP) in 1987, it has undergone significant rehabilitation. This rehabilitation has primarily consisted of repairs to the steel of the movable span and support structure, and the replacement of deteriorated bearings and rivets. In 1990, repairs were performed on the movable span and support steel, and historic bearings and rivets were replaced with newer forms and materials. Also, masonry piers and abutments were repointed and reinforced with steel at that time. Research of existing historic documentation for the bridge revealed that prior coordination with the Connecticut State Historic Preservation Officer (SHPO) had taken place in 2006 relative to the relocation of utilities cables (by the Connecticut Department of Transportation) onto the bridge from the Moses Wheeler Bridge. This prior coordination resulted in the conditional approval of the cable installation, subject to the creation of mitigation documents. No Memorandum of Agreement was created as part of the project mitigation; however, mitigation documents were prepared and submitted to the SHPO by the CTDOT to fulfill SHPO's requirements.

The improvements currently under consideration found the following:

- Alternative I – Short Term Repair would not affect the historic character of the bridge. Consisting of localized repairs to the bridge's superstructure and substructure, and minor repairs and/or upgrades to its mechanical and electrical components, many of these items should be considered maintenance, necessary for the continued functionality of the bridge as it is currently configured.
- Alternatives II, IIIa, IIIb, IVa, & IVb, each include more invasive repair and replacement activities that would affect the historic character of the bridge, and would thus require review by the State Historic Preservation Officer (SHPO). Each of these Alternatives includes the replacement of historic bridge components with modern components that are different in form and structure. At minimum, each of these Alternatives includes the replacement of the bridge's existing high towers with three new monotube towers independent of the bridge structure. Each of these alternatives would likely be considered to have an adverse effect under Section 106 of the National Historic Preservation Act.

When a project alternative is selected, formal coordination with SHPO will be required to formally confirm the determination of effect and any mitigation required as a result. Since extensive documentation of the bridge has already taken place as part of the Moses Wheeler Bridge project, FHI asked SHPO whether additional mitigation may be required. In October of 2010, SHPO advised that demolition of the bridge would require additional mitigation measures. SHPO indicated that the recommended mitigation may consist of "a short documentary video focusing on the economic and engineering constraints faced by early 20th-century rail and bridge engineers and the solutions they chose to allow for a major river crossing while maintaining a navigable river channel", in a format appropriate for a middle school-age audience. As noted, however, no matter which alternative is selected, coordination with SHPO will be necessary, at which time SHPO will provide official determinations.

C.) Summary of Permit Requirements

	Alternative				
Permit/Review	**Alternative I**	**Alternative II**	**Alternative IIIa**	**Alternative IIIb**	**Alternatives IVa and IVb**
U.S. Coast Guard Bridge Permit			†	X	X
U.S. Army Corps of Engineers Programmatic General Permit with CT Addendum		X	X	X	
U.S. Army Corps of Engineers Individual Permit					X
CT DEP Structures, Dredging & Fill		X	X	X	X
CT DEP Tidal Wetlands		†	†	†	X
CT DEP 401 Water Quality Certification		X	X	X	X
CT DEP Flood Management Certification		X	X	X	X
Coastal Consistency Review	X	X	X	X	X
CT DEP Natural Diversity Database Review	X	X	X	X	X
Coordination with U.S. Fish & Wildlife Service and National Marine Fisheries Service	X	X	X	X	X
CT Registration for General Permit for Contaminated Soil and/or Sediment Management					X

X=Permit is likely to be required; †= Permit may be needed.

X. Seismic Assessment

A seismic analysis performed on the structure in accordance with the 2007 AREMA Specifications, Chapter 9 Section 1.5 for general structural parameters and analysis methodology. Although these provisions do not specifically address movable bridges, they are adequate for the purposes of this existing conditions evaluation. It should be noted that the most applicable reference that addresses seismic analysis of movable bridges is the AASHTO Movable Bridge Inspection, Evaluation, and Maintenance Manual (1998). Section 3.4.2.4 of this specification recognizes when movable spans are held in one position (open or closed) for more than 90% of the time, a 50% reduction in the seismic load may be used when evaluating the other position. In the case of the Devon Bridge, the movable span is in the open position much less than 1% of the operating time and the bridge essentially performs as a non-movable structure. Thus seismic analysis of the bridge in the open position was deemed unnecessary.

A.) Methodology and Site Characteristics

AREMA outlines general requirements for seismic analysis of rail structures in Chapter 9 Section 1.3, with the overall objectives of ensuring the safety of trains and minimizing the costs of damage and loss of use of the particular facility. Toward that end, AREMA utilizes a performance criteria based on three Limit States which correspond to three different Ground Motion Levels:

Limit State	Ground Motion Level
Serviceability	1
Ultimate	2
Survivability	3

Each Ground Motion Level subsequently corresponds to an average earthquake return period. The return period is based on a calculated importance factor that considers Immediate Safety, Immediate Value, and Replacement Value. The large size and heavy usage of the bridge result in the return period for each of the Limit States which are at the upper limit of the range for each Ground Motion Level:

Ground Motion Level	Avg. Return Period	Calculated Return Period
1	50-100 year	100
2	200-500 year	500
3	1,000-2,400 year	2,400

The base acceleration coefficient represents site dependent ground motion as a fraction of gravitational constant, g, and is dependent upon the Return Period. Based on the above calculated return periods, the acceleration coefficients defined on the AREMA acceleration coefficient maps for each of the Limit States is: 0.05 for Serviceability (100 year return period), 0.13 for Ultimate (475 year), and 0.25 for Survivability (2,400 year). The CTDOT Bridge Design Manual specifies an acceleration coefficient of 0.16 be used for seismic analysis of bridges within the state. This corresponds roughly to the AREMA defined Ultimate (475 year) Limit State. Thus, the following acceleration coefficients and performance criteria limit states were used for the analysis:

Limit State	Return Period	Acceleration Coefficient
Serviceability	100 year	0.05
Ultimate	500 year	0.16
Survivability	2,400 year	0.25

AREMA Chapter 9 Section 1.4.4.1 defines the Site Coefficient (S) for four different soil types. Based on available information, the soil can be classified as Soil Type 1, with a sand and gravel layer less than 200 feet to bedrock. Thus, the Site Coefficient used for analysis is 1.0.

Based on the expected response of the unreinforced masonry piers, the Response Modification Factor of 1.0 was used based on Section 3.7 of Division 1A of the AASHTO Standard Specifications.

Consistent with previous seismic analyses of this structure, the bridge was analyzed using the Equivalent Lateral Force Procedure, as the bridge consists of a series of single span structures. This method was chosen based on the assumption that the individual spans will respond in their fundamental mode of vibration. The period of each span was calculated independently, and the loads distributed based on specific bearing configurations at each pier. Mass effects of the piers were also included in the analysis.

B.) Results

The bridge responds well for all limit states in the transverse direction. This is expected due to the relative stiffness of the bridge piers in that direction. In the transverse direction, the bridge appears adequate for the Serviceability Limit State. However the analysis indicates global pier instability (overturning) would occur at the Ultimate and Survivability Limit States. Controlling Factors of Safety for pier stability in the longitudinal direction are as follows:

Limit State	Overturning FS	Sliding FS
Serviceability	1.87	7.02
Ultimate	0.79	2.19
Survivability	0.54	1.40

XI. Design Alternatives

A.) Introduction

Stantec developed conceptual rehabilitation and replacement alternatives that partially or fully address deficiencies found during our inspection and analyses. The alternatives were developed based on three service life horizons: 5-7 year, 25-year, and 75-year. The alternatives are segregated as such to allow for decisions regarding the future allocation of funds relative to the short and long term options associated with the Devon Bridge. For Alternatives III and IV, the existing high towers carrying high voltage feeder lines will be replaced with new monotube towers.

The alternatives developed are as follows:

- **Alternative I – Short Term Repair.** This alternative consists of performing minor repairs or replacement of deteriorated members to increase the useful life of the bridge by 5 to 7 years.
- **Alternative II – Rehabilitation.** This alternative involves performing major repairs to both the super- and substructure, replacement of major structural members and systems and construction of additional items to upgrade the useful life of the bridge to approximately 25 years.
- **Alternative IIIa – Partial Superstructure Replacement.** This alternative involves replacing Spans 5, 6, and 7 of the superstructure while rehabilitating Spans 1, 2, 3 and 4, and using the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years.
- **Alternative IIIb – Complete Superstructure Replacement.** This alternative involves replacing the entire superstructure and using the majority of the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years. New movable bridge types that will be investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule. The substructure units that support the movable span types other than the Scherzer rolling lift will require new substructure units.
- **Alternative IVa – Full Replacement with Trusses.** This alternative involves replacing both the superstructure and the substructure to increase the useful life of the bridge to 75+ years. The superstructure consists of thru trusses similar to existing. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule.
- **Alternative IVb – Full Replacement with Deck Girders.** This alternative involves replacing both the superstructure and the substructure to increase the useful life of the bridge to 75+ years. The superstructure consists of a deck girder system. New movable bridge types investigated with this alternative are vertical lift, Scherzer rolling lift, and heel trunnion bascule.

B.) Site Conditions

The Devon Bridge is located approximately 90 feet north of the existing Moses Wheeler Bridge (Interstate 95). Along the west bank, retail and commercial parking areas predominating the area. A marina is located on the west bank just south of the Moses Wheeler Bridge, and uses the parking area for boat storage. The east bank is less developed, with the Housatonic River Boat Launch (CTDEP) immediately to the southeast of the bridge, and undeveloped land immediately to the northeast. The Waterbury Branch connects to Track 3 of the New Haven Line approximately 110 feet east of the Devon Bridge’s easterly abutment. The Moses Wheeler Bridge is currently being widened to the north, which ultimately will reduce the horizontal clearance between the two structures to approximately 40 feet.

1. Contractor Access

The Housatonic River Boat Launch at the southeast corner of the bridge would provide an ideal location for a contractor staging area due to its immediate proximity to the bridge and access to the water. Construction access to the bridge could be staged off of barges or a temporary work platform extending to the piers from the boat launch. The contractor may also be able to access the west side of the bridge from the existing parking/marina boat storage area.

Access is also available to the top side of the bridge from the east. An at-grade maintenance crossing exists to connect the boat launch area with the Metro-North maintenance tower approximately 600 feet east of the bridge. In addition, there is a maintenance spur that connects to Track 4 at this location that could be used for staging rail mounted equipment.

Photo XI-1: DEP boat launch at southeast corner

2. Horizontal Constraints

The two High Towers immediately to the east and west of the bridge present significant lateral horizontal constraints. The high towers would require modification or replacement should any lateral shift of the tracks be contemplated. The rail bed is located on an embankment in the immediate vicinity of the bridge approaches; although not currently flagged, any lateral shift of the tracks would likely impact wetlands due to additional fill required. To the south, the Moses Wheeler Bridge will be reconstructed approximately 50 feet closer than existing, providing a lateral clearance of approximately 40 feet.

Photo XI-2: Moses Wheeler Bridge to left, Devon Bridge to right

Between Piers 3 and 4, the navigation channel of the Housatonic River at the Devon Bridge is 83 feet minimum. This horizontal clearance is not reduced with any of the alternatives investigated. The new Moses Wheeler Bridge, when completed, will provide horizontal clearance of 100 feet at the navigation channel.

3. Vertical Constraints

The primary vertical control is the grade of the existing tracks; any vertical shift of the track elevation at the bridge would require significant approach work to achieve properly. The existing vertical clearance of the bridge in closed position over the navigation channel at high tide is 19 feet; this clearance is not reduced with any of the alternatives investigated. The new Moses Wheeler Bridge, when completed, will provide approximately 69 feet of vertical clearance above Mean High Water within the navigation channel. This vertical clearance is used as a minimum for alternatives that consider replacement of the existing movable bridge. Note that a non-movable bridge at this location would be cost prohibitive and impractical due to a minimum of two miles of track vertical realignment with associated impacts to catenaries, roadway overpasses and underpasses, as well as long term staging operations associated with the vertical relocation.

C.) Alternative I – Short Term Repairs

1. Description

This alternative consists of performing localized repairs to the structural steel superstructure, minor repairs to the substructure, and minor repairs and/or upgrades to the mechanical and electrical components. A number of steel stringers exhibit edge and pitting losses at the flanges at all truss spans which contribute to the reduction in load capacity, in particular those stringers below Track 1 (Stringers S-3 and S-4). At several locations, localized repairs to truss members will be performed. In addition, a number of secondary members (lateral bracing) will be repaired as required. Deteriorated rivets and bolts will be replaced, and any missing rivets/bolts will be installed. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking.

This alternative does not include repairs to the mechanical and electrical systems, as these repairs are generally performed by MNR maintenance forces. This maintenance should include lubricating all moving components, tightening loose fasteners and repairing failed or cracked welds. Brake shoe contact should be adjusted at all locations, and the tips of the south rack and pinion of the north bascule leaf will be ground down to eliminate tooth bottoming.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair tread, railing and railing posts will also be repaired or replaced. An adequate ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

2. Sequencing and Duration

The majority of the work contemplated with this alternative would be completed while the tracks are in service, or during short term track outages. Repairs to primary members (trusses, floorbeams, stringers) would be completed during short term track outages to reduce loads and vibration occurring at the repair site.

Repairs to mechanical and electrical control components would be completed during times of off-peak marine traffic (mid-winter). The staging of such repairs would occur such that the spans could be opened on a maximum of 24 hours notice.

The duration of construction for all repairs will depend on availability and duration of track outages. Provided these outages occur with reasonable frequency and durations, these repairs could be completed in approximimately 10 months.

3. Constructability

Structural components located below the track level, such as floorbeams, stringers, deck giders, bottom lateral bracing, and bottom chord members, would be accessed from the river via barges. Access from the barges could be supplemented by temporary scaffolding mounted to the underside of the bridge at specific repair locations.

Access to structural components above track level, such as upper chord members, top lateral bracing, portals, diagonals, verticals, and hangers, would be accessed from track mounted bucket trucks.

4. Railroad Operational Impacts

Impacts to railroad operations resulting from work proposed in this alternative would be minimal. Short term track outages would be required during off peak times, typically with only one track removed from service at a time. Trains from the Waterbury Branch complicate track outages on Track 3, and significantly limits the effective window of working time. Scheduling for a longer term track outage may be required for Track 3 to provide the contractor with realistic working windows.

5. Marine Operational Impacts

Impacts to marine operations would be minimal, and would occur while the contractor is accessing the underside of the bridge, providing repairs to the substructure units, or repairing the fender system. Impacts would consist of the contractor maneuvering a barge or work platform as required for access. The barge would be able to be moved out of the navigation channel within 24 hours notice.

6. Utility Impacts

Impacts to utilities with this alternative will be minor, consisting primarily of protection of utilities in localized areas as specific repairs are completed.

7. Right of Way Impacts

No permanent right of way impacts are required for this alternative, as the existing bridge is located within the State of Connecticut Right of Way. Temporary construction easements may be required for the contractor's staging area and access to the bridge.

8. Environmental Impacts

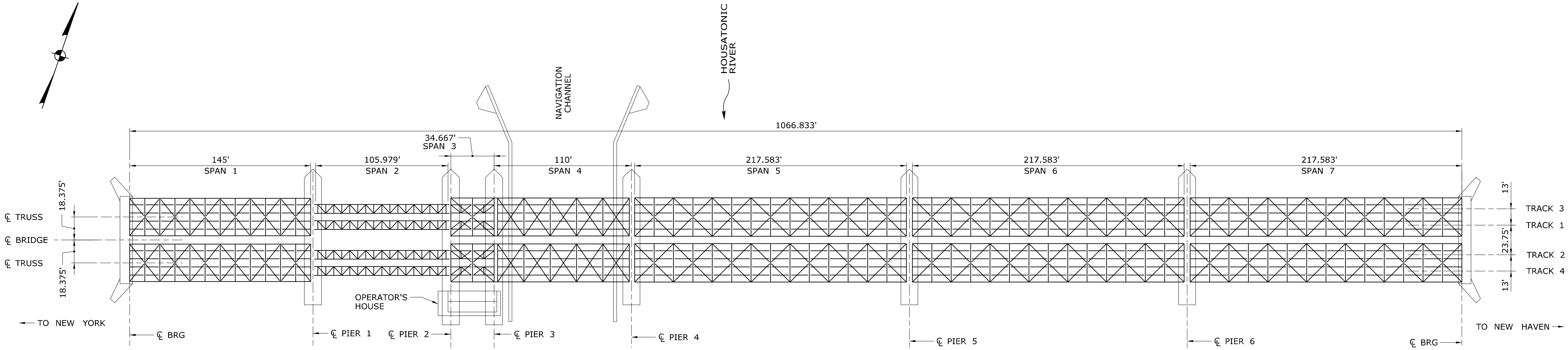
No significant environmental impacts are associated with this alternative. Aside from normal environmental construction measures, any work by the contractor on the structural steel would require proper measures to prevent lead paint from entering the river or upland areas. The contractor would be required to follow Best Management Practices.

9. Historical Impacts

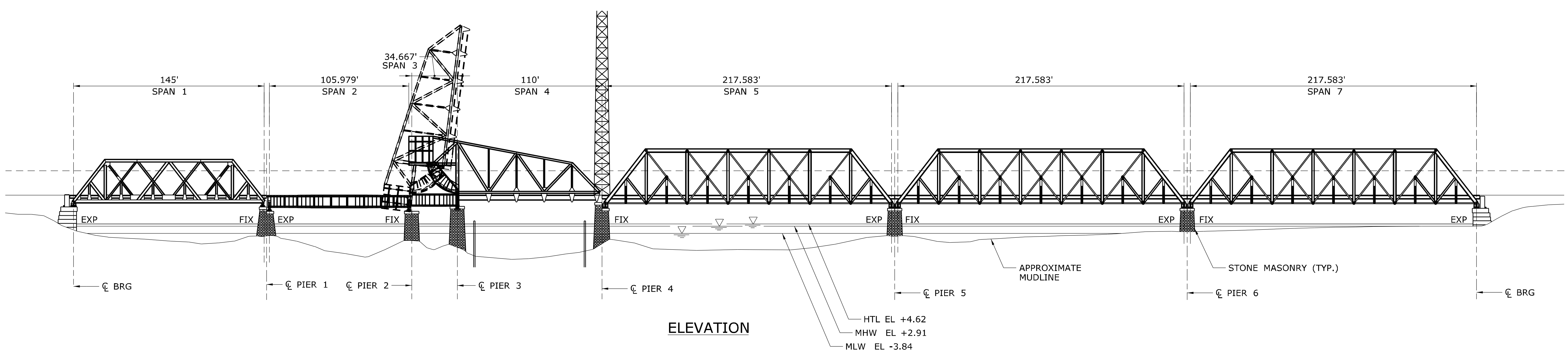
No significant historical impacts are associated with this alternative.

10. Hydraulic Impacts

No significant hydraulic impacts are associated with this alternative.



FRAMING PLAN



ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: FRAMING PLAN	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-100

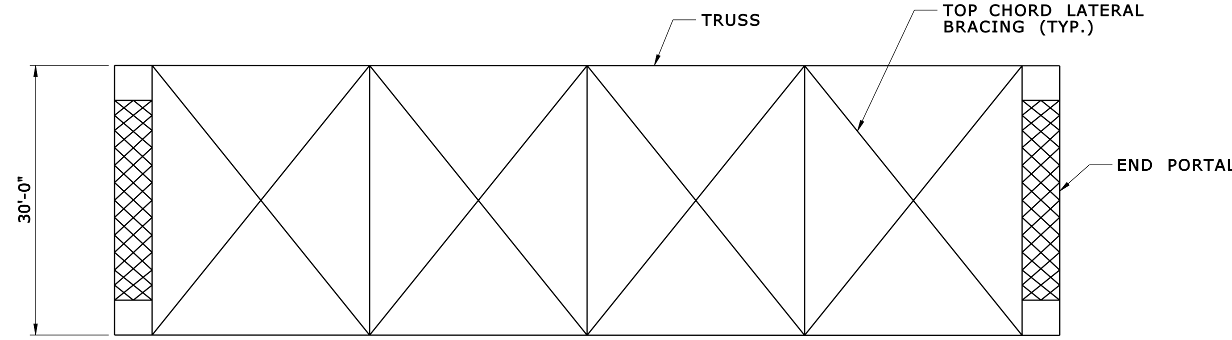
s-100.dgn 10/1/2010 2:07:50 PM

FRAMING LEGEND:

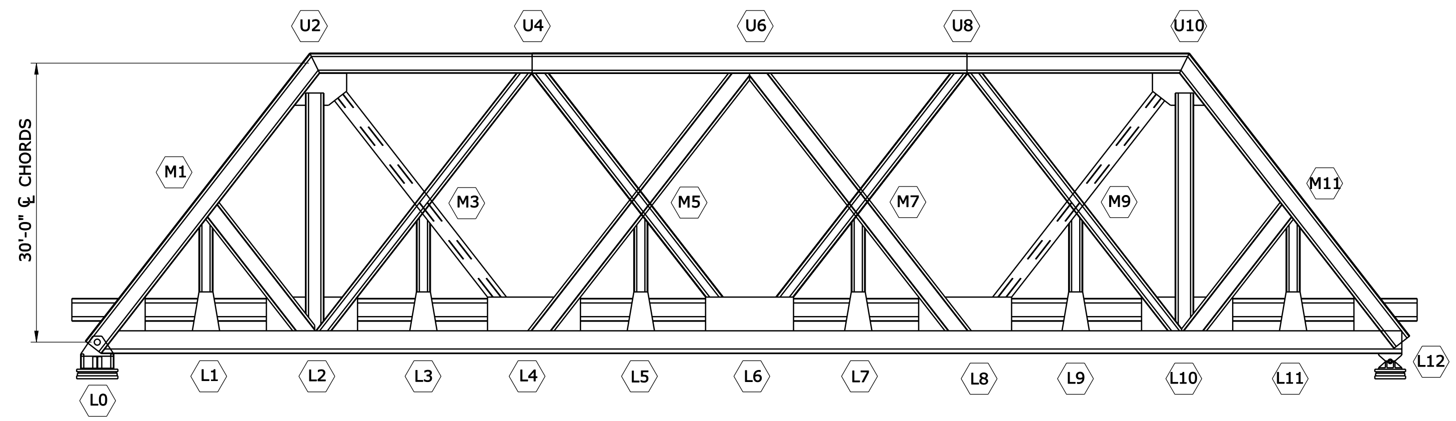
- U2 PANEL POINT NUMBER
- FB0 FLOORBEAM
- S1 STRINGER

REPAIR LEGEND:

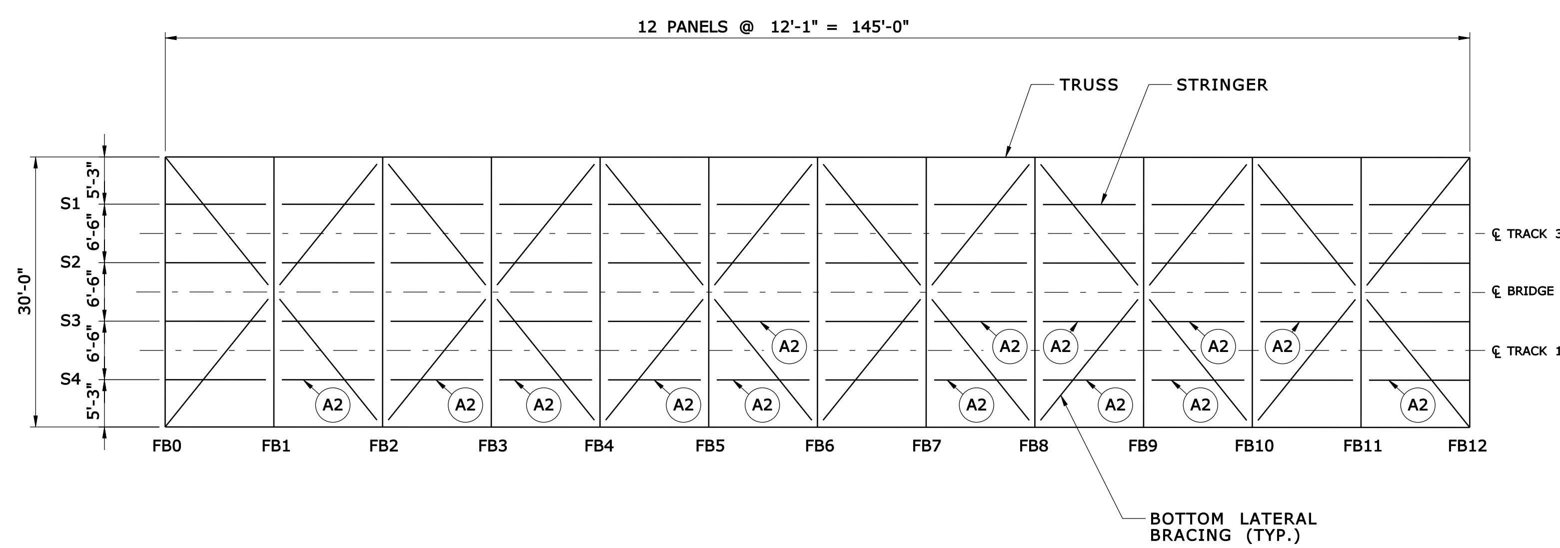
- A1 STRINGER BOTTOM FLANGE REPAIR
- A2 STRINGER TOP FLANGE REPAIR
- RXX RIVET REPLACEMENT WITH H.S. BOLTS (XX INDICATES APPROXIMATE NUMBER OF RIVETS TO BE REPLACED)



TOP CHORD FRAMING PLAN



TRUSS ELEVATION



BOTTOM CHORD FRAMING PLAN

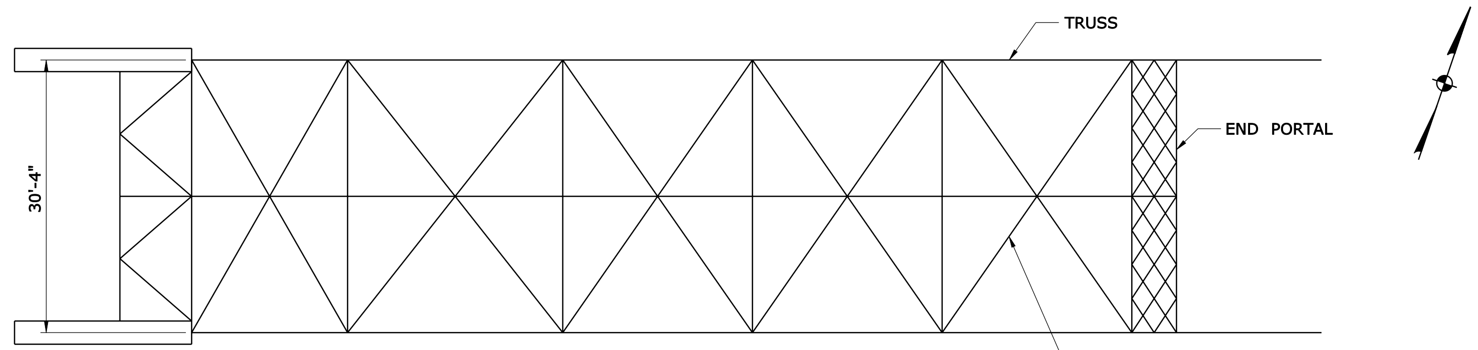
STATE PROJECT NO.: 301-099	DRAWING TITLE: SCHEMATIC REPAIRS - SPAN 1 NORTH	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 3/32" = 1'-0" FIGURE: S-101N

FRAMING LEGEND:

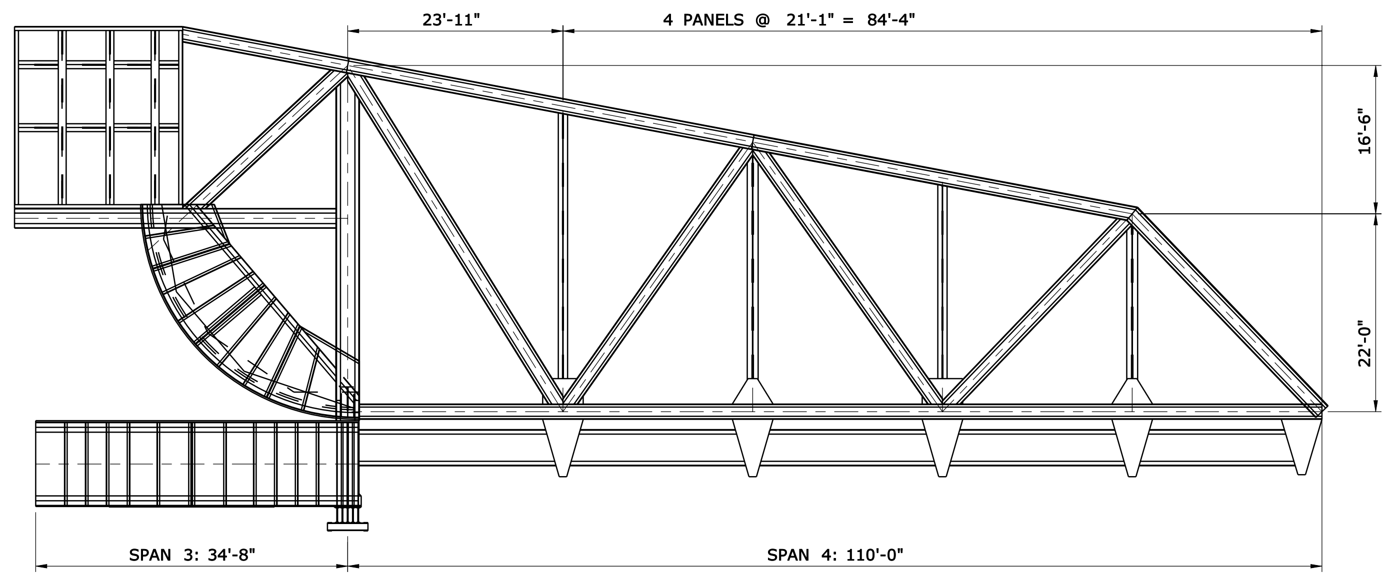
- U2 PANEL POINT NUMBER
- FB0 FLOORBEAM
- S1 STRINGER

REPAIR LEGEND:

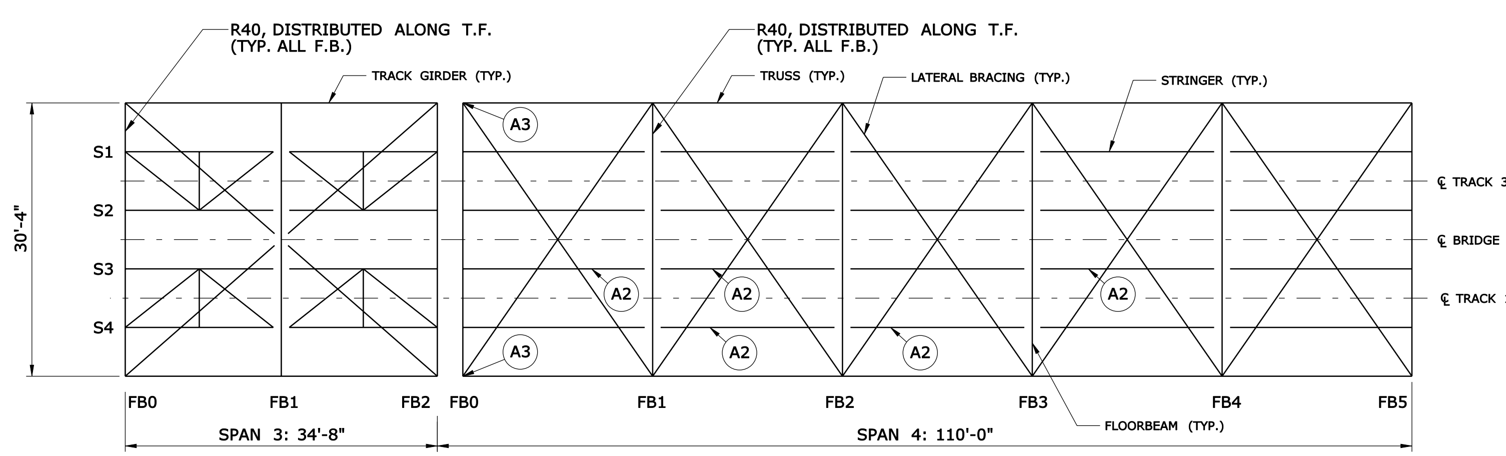
- A1 STRINGER BOTTOM FLANGE REPAIR
- A2 STRINGER TOP FLANGE REPAIR
- A3 FLOORBEAM WEB REPAIR
- RXX RIVET REPLACEMENT WITH H.S. BOLTS (XX INDICATES APPROXIMATE NUMBER OF RIVETS TO BE REPLACED)



TOP CHORD FRAMING PLAN

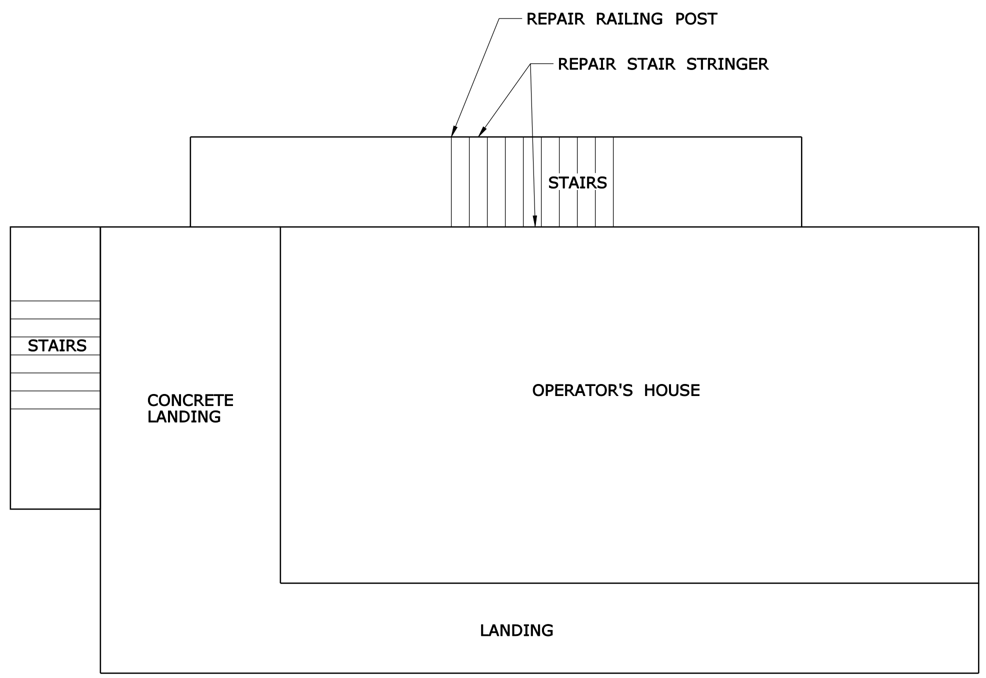


TRUSS ELEVATION

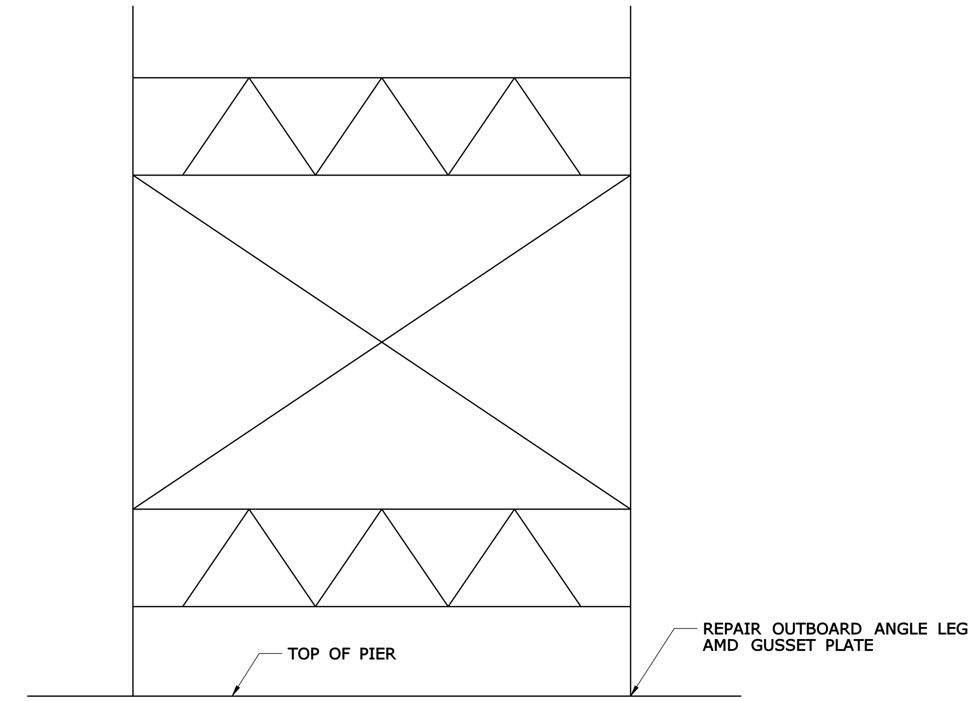


BOTTOM CHORD FRAMING PLAN

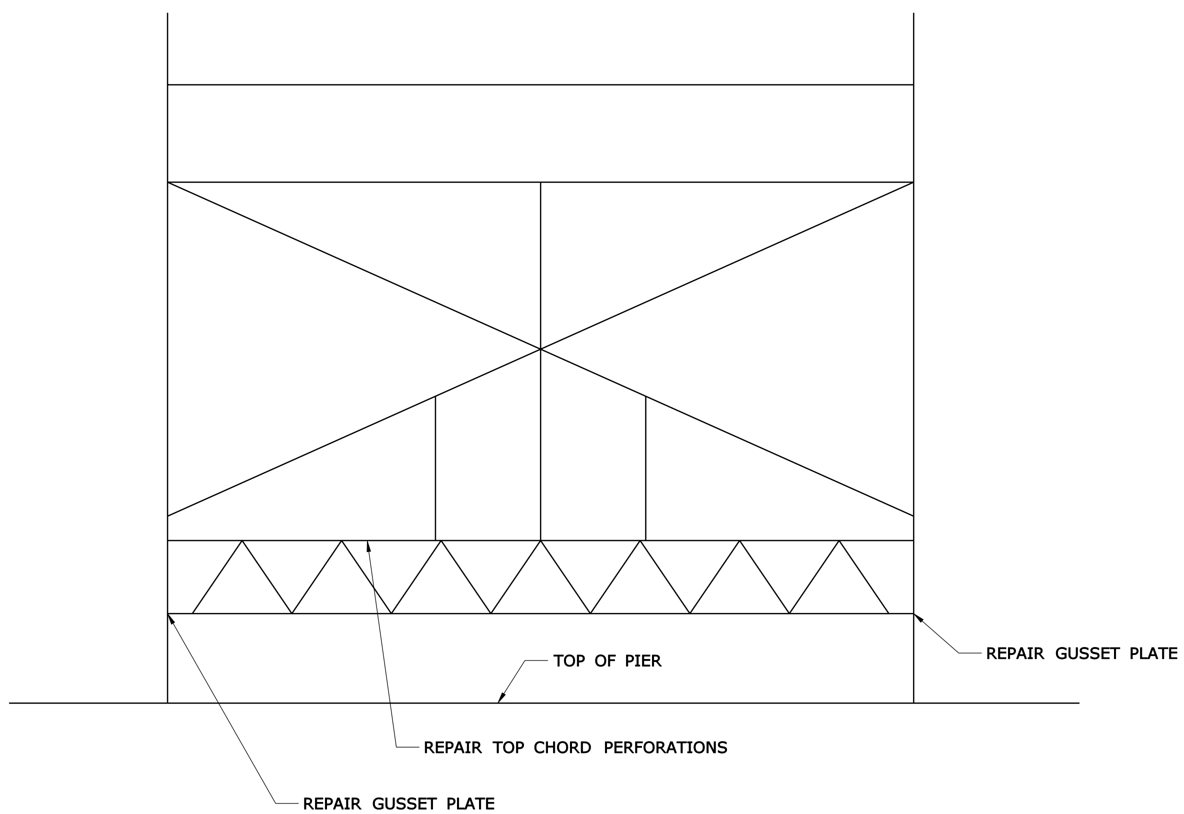
STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	SCHEMATIC REPAIRS - SPANS 3 & 4 NORTH	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE: 3/32" = 1'-0"
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	FIGURE: S-104N



OPERATOR'S HOUSE - PLAN



OPERATOR'S HOUSE FRAMING - EAST TRUSS - WEST ELEVATION



OPERATOR'S HOUSE FRAMING - NORTH TRUSS - SOUTH ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVE I - OPERATOR'S HOUSE	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: N.T.S. FIGURE: S-104OH

FRAMING LEGEND:

U2 PANEL POINT NUMBER

FB0 FLOORBEAM

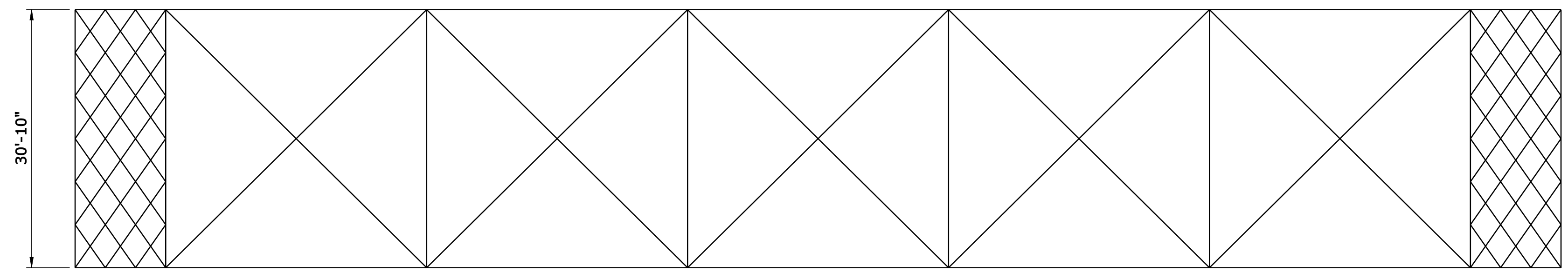
S1 STRINGER

REPAIR LEGEND:

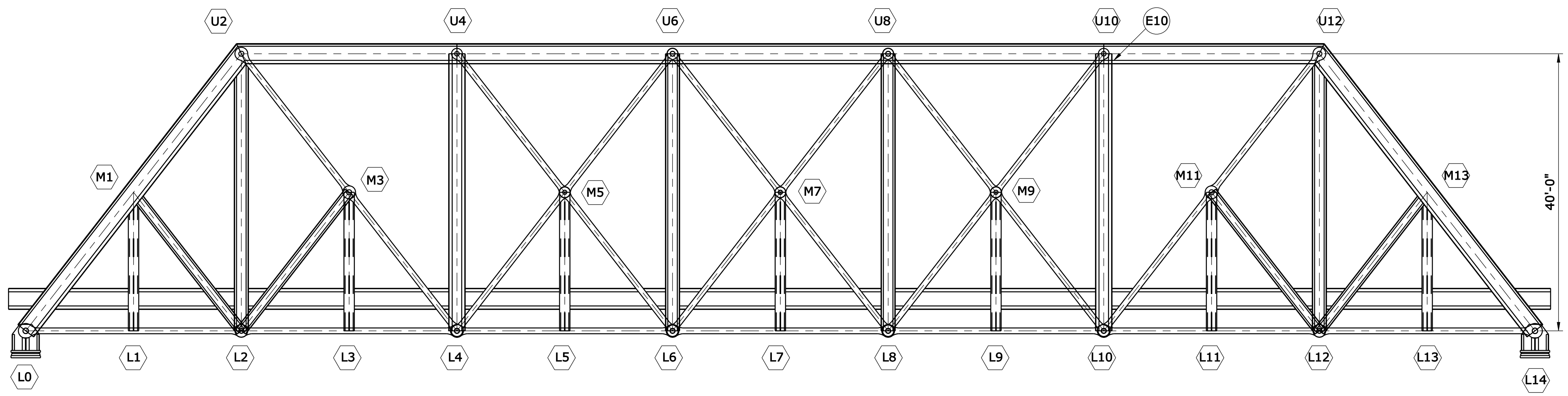
A1 STRINGER BOTTOM FLANGE REPAIR

A2 STRINGER TOP FLANGE REPAIR

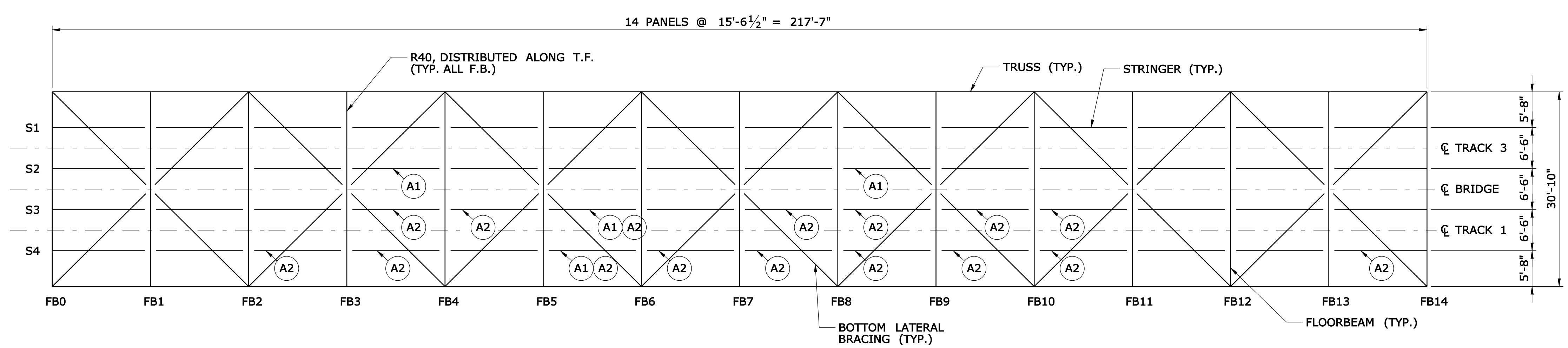
RXX RIVET REPLACEMENT WITH H.S. BOLTS (XX INDICATES APPROXIMATE NUMBER OF RIVETS TO BE REPLACED)



TOP CHORD FRAMING PLAN



TRUSS ELEVATION



BOTTOM CHORD FRAMING PLAN

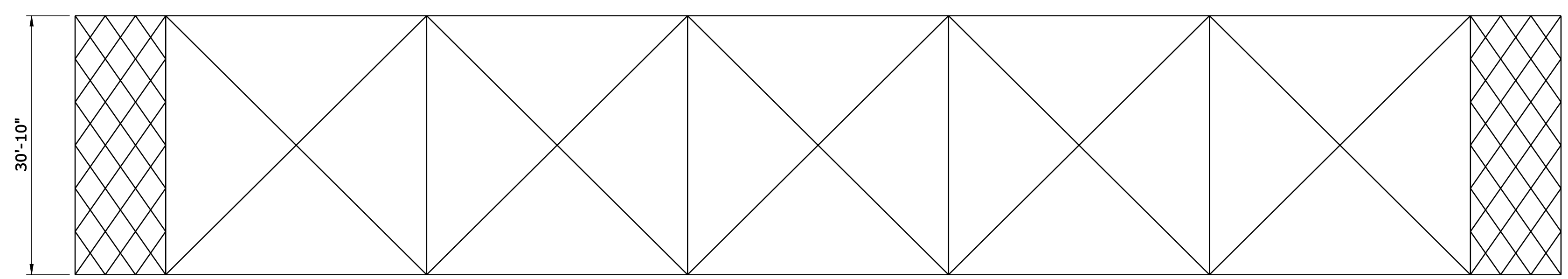
STATE PROJECT NO.: 301-099	DRAWING TITLE: SCHEMATIC REPAIRS - SPAN 5 NORTH	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 3/32" = 1'-0"
		FIGURE: S-105N

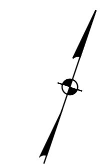
FRAMING LEGEND:

- U2 PANEL POINT NUMBER
- FB0 FLOORBEAM
- S1 STRINGER

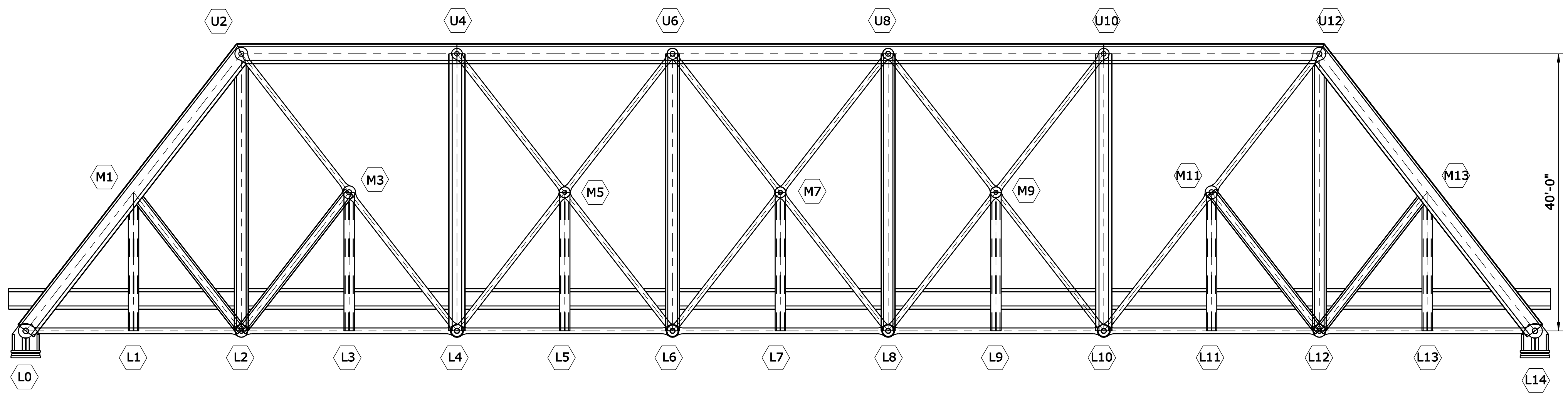
REPAIR LEGEND:

- A1 STRINGER BOTTOM FLANGE REPAIR
- A2 STRINGER TOP FLANGE REPAIR
- RXX RIVET REPLACEMENT WITH H.S. BOLTS (XX INDICATES APPROXIMATE NUMBER OF RIVETS TO BE REPLACED)

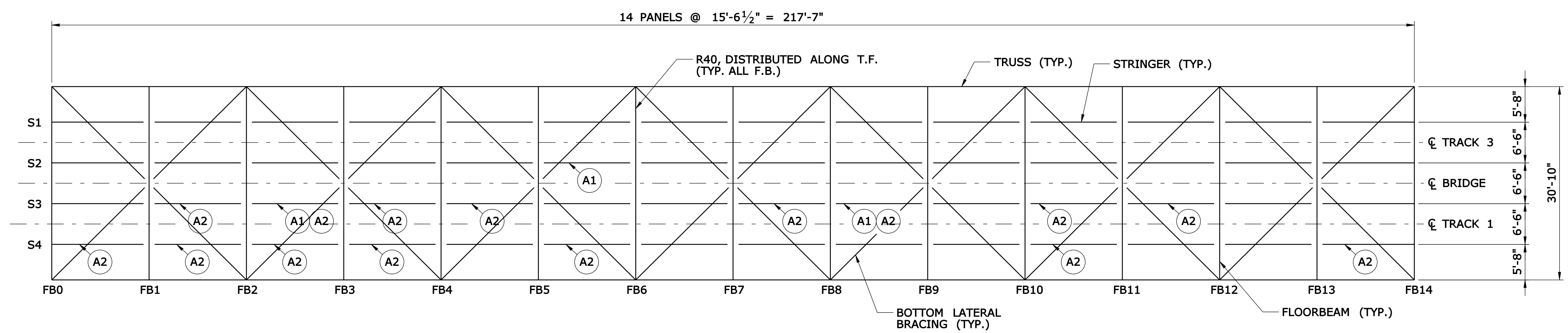




TOP CHORD FRAMING PLAN



TRUSS ELEVATION



BOTTOM CHORD FRAMING PLAN

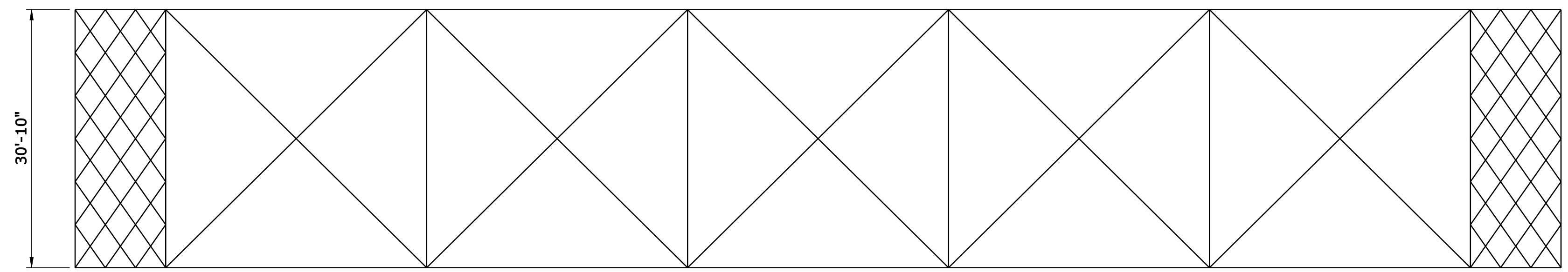
STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	SCHEMATIC REPAIRS - SPAN 6 NORTH	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE:
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	3/32" = 1'-0"
		FIGURE: S-106N

FRAMING LEGEND:

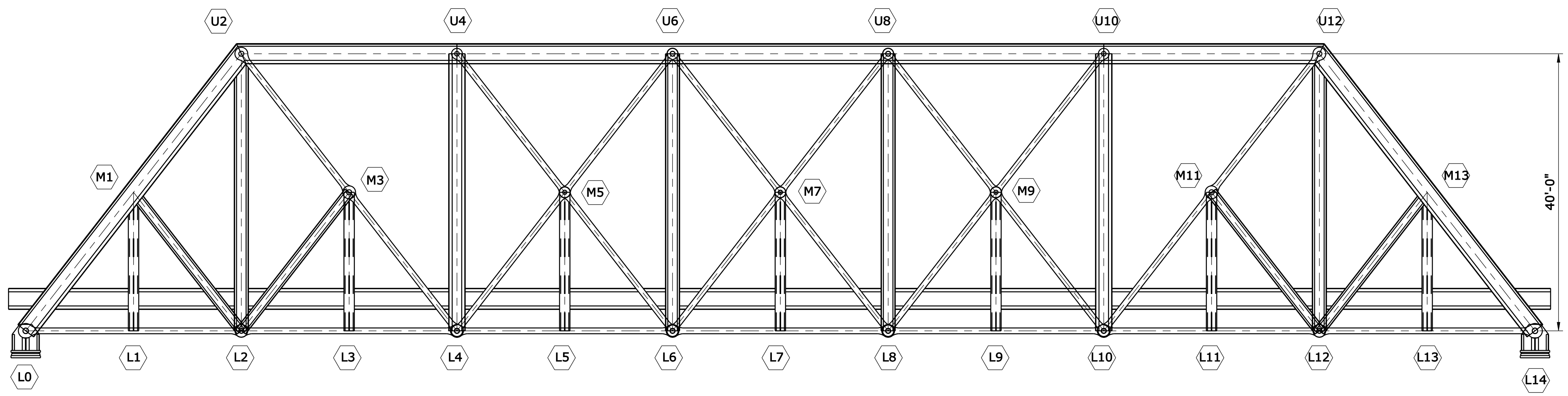
- U2 PANEL POINT NUMBER
- FB0 FLOORBEAM
- S1 STRINGER

REPAIR LEGEND:

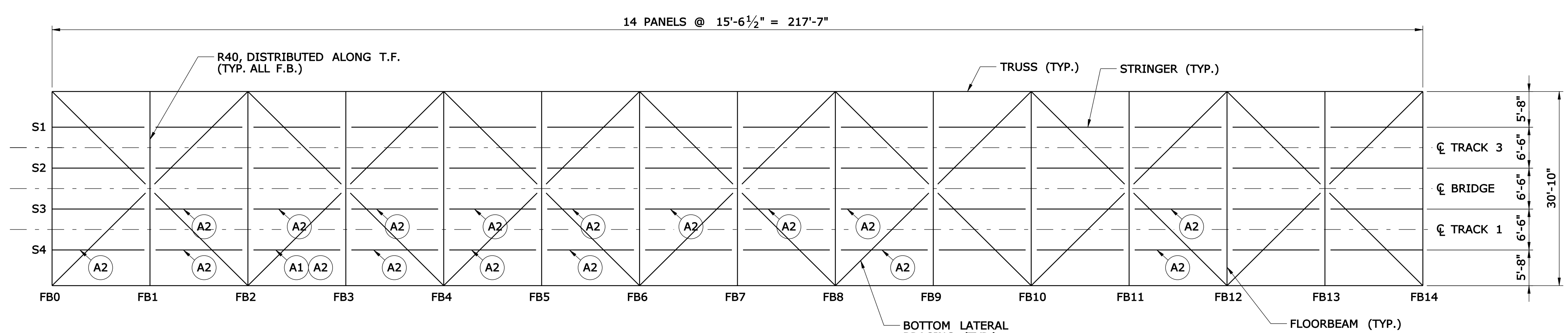
- A1 STRINGER BOTTOM FLANGE REPAIR
- A2 STRINGER TOP FLANGE REPAIR
- RXX RIVET REPLACEMENT WITH H.S. BOLTS (XX INDICATES APPROXIMATE NUMBER OF RIVETS TO BE REPLACED)



TOP CHORD FRAMING PLAN



TRUSS ELEVATION



BOTTOM CHORD FRAMING PLAN

STATE PROJECT NO.: 301-099	DRAWING TITLE: SCHEMATIC REPAIRS - SPAN 7 NORTH		DATE: JUNE 2010
	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)		SCALE: 3/32" = 1'-0"
CITY/TOWN: MILFORD/STRATFORD			FIGURE: S-107N

D.) Alternative II – Rehabilitation

1. Description

This alternative consists of performing substantial repairs to the structural steel superstructure, repairs to the substructure, and a major upgrade to the mechanical and electrical components. Many of the built up stringers, primarily below Track 1 (Stringers 3 and 4), will be replaced with wide flange members. Floorbeams will be repaired as required at Spans 1, 3, 4, 5, 6, and 7. Repairs to remaining truss and secondary members considered in Alternative I will be completed. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking. A scour monitoring system will be installed at Piers 2, 3, 4, and 5. Signals and communications facilities will be upgraded as required during rehabilitation; however no catenary work will be performed. This alternative will also include replacement of the monorails and track repair work as required to facilitate repair procedures.

An on-site standby generator will be installed to provide power to the bridge control system and Operator's House during power outages. All other repairs noted during Alternative I will be completed as well.

The major components of the mechanical system will be replaced, including the racks and pinions, various drive gearsets, span locks, and live load shoes and centering devices. In addition, the air buffers will be removed.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair tread, railing and railing posts will also be repaired or replaced. An adequate ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

All structural steel, including Spans 1 through 7, machinery, and Operator's House support, will be blast cleaned and painted.

The existing high towers will be replaced with three new monotube towers independent of the bridge structure.

2. Sequencing and Duration

The majority of the work contemplated with this alternative would be completed during a two track outage of the railroad, whereby each of the two independent truss systems is rehabilitated separately. Taking tracks out of service is necessary to allow for efficient contractor operations, as well as to reduce the loads on the superstructure components during repair operations.

Repairs to mechanical and electrical control components would be completed during times of off-peak marine traffic (mid-winter). The staging of such repairs would occur such that the spans could be opened on a maximum of 24 hours notice.

Stage 1

Stage 1 will consist of the rehabilitation of the northerly truss system, with long term track outages to Tracks 1 and 3. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

After the tracks are taken out of service, work would begin on repairs to steel elements of the superstructure. The replacement of individual stringers within the truss spans could occur from below to eliminate the need to remove rails and ties on the bridge. However efficiency could be obtained by removing the rails and track and accessing the stringers from the track level. Repairs to floorbeams would occur as the stringers are removed and being replaced.

Upgrades to the mechanical system serving the north leaf will occur concurrent with other structural repairs.

The duration of Stage 1 will last approximately 6 to 12 months.

Stage 2

Stage 2 will consist of the rehabilitation of the southerly truss system, with long term track outages to Tracks 2 and 4. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

As with Stage 1, work would begin on repairs to steel elements of the truss after the tracks are taken out of service. The replacement of individual stringers within the truss spans could occur from below to eliminate the need to remove rails and ties on the bridge. However efficiency could be obtained by removing the rails and track and accessing the stringers from the track level. Repairs to floorbeams would occur as the stringers are removed and being replaced.

Upgrades to the mechanical system serving the south leaf will occur concurrent with other structural repairs.

The duration of Stage 2 will last approximately 6 to 12 months.

Stage Independent Work

Other work associated with this alternative, such as repairs to the Operator's House, associated framing and the substructure, can occur concurrent with the above Stage Construction work.

3. **Constructability**

Structural components located below the track level, such as floorbeams, stringers, deck girders, bottom lateral bracing, and bottom chord members, would be accessed from the river via barges or work platforms constructed in the river. Access from barges could be supplemented by temporary scaffolding mounted to the underside of the bridge at specific repair locations.

Access to structural components above track level, such as upper chord members, top lateral bracing, portals, diagonals, verticals, and hangers, would be accessed from track mounted bucket trucks.

The replacement of the truss pins and eyebars will prove to be a difficult task at best. The lack of redundancy inherent in the truss systems requires that temporary load paths be created to transfer loads between connected members as each pin and/or eyebar is removed for replacement.

Several ways in which the pins and eyebars could be replaced were investigated, including these conceptual methods:

- Temporarily supporting the truss with falsework in the river;
- Extended track outages with temporary structural steel girders to support dead loads; and
- Temporarily replacing tension members of truss with cables or other members.

However, the sheer number of truss pins (252) and eyebars (1200) that help form the trusses of Spans 5, 6, and 7 render it nearly impossible to cost effectively replace them without significant impacts to rail operations. Further, the lack of expertise by Contractors in performing this work will likely lead to exceptionally high construction bids.

4. **Railroad Operational Impacts**

Impacts to railroad operations resulting from work proposed in this alternative would be substantial, and will result in impacts to operations between CP 255 and CP 261 at a minimum. A minimum of four (4) miles of two track service is necessary unless a new series of interlockings are added between CP 255 and CP 261. This two track service will affect the operational flexibility of the railroad. By only allowing for one train in each direction, peak hour service, especially as relates to express and local trains, would be affected.

Short term track outages may be utilized during off peak times, typically with only one track removed from service at a time. Trains from the Waterbury Branch complicate track outages on Track 3, and significantly limits the effective window of working time. Scheduling for a longer term track outage may be required for Track 3 to provide the contractor with realistic working windows.

Temporary platforms would be required at one station at a minimum during one of the stages of construction.

Specific operational impacts to the railroad would depend on the staging alternative chosen. Refer to Section V.B of this report.

5. Marine Operational Impacts

Impacts to marine operations would be minimal, and would occur while the contractor is accessing the underside of the bridge, providing repairs to the substructure units, or repairing the fender system. Impacts would consist of the contractor maneuvering a barge or work platform as required for access. The barge would be able to be moved out of the navigation channel within 24 hours notice.

6. Utility Impacts

TBD

7. Right of Way Impacts

No permanent right of way impacts are required for this alternative, as the existing bridge is located within the State of Connecticut Right of Way. Temporary construction easements may be required for the contractor's staging area and access to the bridge.

8. Environmental Impacts

No significant environmental impacts are associated with this alternative. Aside from normal environmental construction measures, any work by the contractor on the structural steel would require proper measures to prevent lead paint from entering the river or upland areas. The contractor would be required to follow Best Management Practices. Cleaning and painting of the existing structural steel will require adequate containment and worker protection measures due to the lead paint detected in the various samples taken.

9. Historical Impacts

No significant historical impacts are associated with this alternative.

10. Hydraulic Impacts

No significant hydraulic impacts are associated with this alternative.

E.) Alternative IIIa – Partial Superstructure Replacement

1. Description

This alternative consists of replacing Spans 5, 6 and 7 with new truss structures, completing a rehabilitation of the Spans 1, 2, 3, and 4, performing repairs to the substructure, and completing a major upgrade of the mechanical and electrical components. Many of the built up stringers, primarily below Track 1 (Stringers 3 and 4), will be replaced with wide flange members at Spans 1, 3 and 4. The deck girders of Span 2 will be repaired as in Alternative I. The new spans at Spans 5, 6, and 7 would be similar to existing, with the exception that the truss members would be rolled steel sections and would be connected using bolted gusset plates. The floor system would be a floorbeam and stringer configuration similar to existing. Repairs to remaining truss and secondary members at Spans 1 through 4 considered in Alternative I will be completed. The masonry substructure will be repointed where existing mortar has failed or exhibits cracking. A scour monitoring system will be installed at Piers 2, 3, 4, and 5. The signals and communication system will be replaced, as will the catenary and all track on the bridge.

An on-site standby generator will be installed to provide power to the bridge control system and Operator's House during power outages. All other repairs noted during Alternative I will be completed as well.

The major components of the mechanical system will be replaced, including the racks and pinions, various drive gearsets, span locks, and live load shoes and centering devices. In addition, the air buffers will be removed.

Deteriorated portions of the structural steel supporting the Operator's House will be repaired. Deteriorated sections of stair tread, railing and railing posts will also be repaired or replaced. An adequate ventilation system will be installed in the restroom to ventilate fumes from waste incineration.

All existing structural steel, including Spans 1 through 4, machinery, and Operator's House support, will be blast cleaned and painted. New structural steel will be painted or galvanized.

The existing high towers will be replaced with three new monotube towers independent of the bridge structure.

2. Sequencing and Duration

The majority of the work contemplated with this alternative would be completed during a two track outage of the railroad, whereby each of the two independent truss systems are replaced and rehabilitated separately. Taking tracks out of service is necessary to replace the existing trusses with new. The duration of the two track outage depends highly on the method of construction chosen for the replacement of trusses at Spans 5, 6, and 7. See Item 3 – Constructability below.

Repairs to mechanical and electrical control components would be completed during times of off-peak marine traffic (mid-winter). The staging of such repairs would occur such that the spans could be opened on a maximum of 24 hours notice.

<u>Stage 1</u>

Stage 1 will consist of work on the northerly truss system, with long term track outages to Tracks 1 and 3. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

After the tracks are taken out of service, work would begin on removing and replacing Spans 5, 6 and 7, along with repairs to steel elements of the superstructure to remain. Access to below track repair areas would be from above, as the rail and ties will be removed already for replacement of Spans 5, 6, and 7. Repairs to floorbeams would occur as the stringers are removed and being replaced.

Upgrades to the mechanical system serving the north leaf will occur concurrent with other structural repairs.

The duration of Stage 1 will last approximately 12-18 months. The duration of construction will depend highly on the method the contractor uses to install the new trusses at Spans 5, 6, and 7. If these trusses are constructed off-site and jacked/lifted into position, the duration for this portion will be considerably less than if the trusses are constructed in place using falsework.

Stage 2

Stage 2 will consist of the rehabilitation of the southerly truss system, with long term track outages to Tracks 2 and 4. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

As with Stage 1, work would begin on replacement of Spans 5, 6 and 7 and repairs to steel elements after the tracks are taken out of service. Access to below track repair areas would be from above, as the rail and ties will be removed already for replacement of Spans 5, 6, and 7. Repairs to floorbeams would occur as the stringers are removed and being replaced.

Upgrades to the mechanical system serving the south leaf will occur concurrent with other structural repairs.

The duration of Stage 2 will last approximately 12-18 months. The duration of construction will depend highly on the method the contractor uses to install the new trusses at Spans 5, 6, and 7. If these trusses are constructed off-site and jacked into position, the duration for this portion will be considerably less than if the trusses are constructed in place using falsework.

Stage Independent Work

Other work associated with this alternative, such as repairs to the Operator's House, associated framing and the substructure, can occur concurrent with the above Stage Construction work.

This section will be expanded later upon acceptance of the two track outage railroad sequencing scenario.

3. Constructability

At Spans 1, 2, 3, and 4:

Structural components located below the track level, such as floorbeams, stringers, deck girders, bottom lateral bracing, and bottom chord members, would be accessed from the river via barges or from a temporary work platform. Access from the barges could be supplemented by temporary scaffolding mounted to the underside of the bridge at specific repair locations.

Access to structural components above track level, such as upper chord members, top lateral bracing, portals, diagonals, verticals, and hangers, would be accessed from track mounted bucket trucks.

At Spans 5, 6 and 7:

The replacement of the existing trusses could proceed using several methods. The most basic construction option is to construct falsework below the existing trusses to facilitate disassembly of the existing truss and construction of the new truss. Falsework supports would extend to the riverbed, and would remain in place until all three trusses have been removed and new trusses installed.

Alternatively, the trusses may be able to be floated in to position on barges, and jacked into position from the barges. For this scenario, the trusses would be constructed either off site, assembled nearby either upstream or downstream of the bridge, or a combination of both. The widening of the Moses Wheeler Bridge reduces the clearance between the two bridges; however there will be adequate room for a new truss to be maneuvered between the two bridges.

The operation would consist of a barge being floated into place under the existing truss. Once the barge was anchored to the riverbed and tracks cut between spans, a system of falsework and hydraulic jacks would be used to raise the bridge off of its bearings. When adequate vertical clearance is obtained, the truss would be translated laterally until clear of the piers. With the existing truss removed, any preparation required for the new trusses would occur, such as bridge seat repairs and bearing installation. With the bridge prepared, the new truss would then be floated into position on a barge, jacked up to provide clearance over the piers, translated laterally into place, and lowered onto the existing substructure. The contractor would need to account for tidal fluctuations during the removal and installation operations.

If the new trusses are constructed off site, the northerly trusses would be floated into position under I-95 and through the opened bascule of the Devon Bridge via the navigation channel. This operation becomes more complicated for installation when transporting the southerly trusses, as the final pier sizes for the Moses Wheeler Bridge have not been established. There will be ample horizontal clearance between I-95 piers at the navigation channel; however the fender system would need to be removed first. Further investigation is required to determine if there is adequate clearance for the trusses to be maneuvered under other spans of the Moses Wheeler Bridge.

The southerly trusses could also be constructed on site, on the east bank in the area between the Moses Wheeler Bridge and the Devon Bridge, and then launched west into the Housatonic River and into place.

Removal and installation of Span 7 trusses cannot be accomplished using a barge due to the shallow water depths under that span, and a combination of falsework and/or cranes would be required to disassemble existing and install new trusses.

4. Railroad Operational Impacts

Impacts to railroad operations resulting from work proposed in this alternative would be substantial, and will result in impacts to operations between CP 255 and CP 261 at a minimum. A minimum of four (4) miles of two track service is necessary unless a new series of interlockings are added between CP 255 and CP 261. This two track service will affect the operational flexibility of the railroad. By only allowing for one train in each direction, peak hour service, especially as relates to express and local trains, would be affected.

Short term track outages may be utilized during off peak times, typically with only one track removed from service at a time. Trains from the Waterbury Branch complicate track outages on Track 3, and significantly limits the effective window of working time. Scheduling for a longer term track outage may be required for Track 3 to provide the contractor with realistic working windows.

Temporary platforms would be required at one station at a minimum during one of the stages of construction.

Refer to Section V.B of this report for additional discussion of staging and impacts.

5. Marine Operational Impacts

At Spans 1, 2, 3, and 4:

Impacts to marine operations would be minimal, and would occur while the contractor is accessing the underside of the bridge, providing repairs to the substructure units, or repairing the fender system. Impacts would consist of the contractor maneuvering a barge or work platform as required for access. The barge would be able to be moved out of the navigation channel within 24 hours notice.

At Spans 5, 6 and 7:

Impacts to marine operations will depend on the truss demolition and alternative chosen for Spans 5, 6, and 7. If temporary falsework is used to provide support, the waterway under these spans would be unusable to marine traffic.

Overall, the impacts to marine operations would be minimal. The navigation channel would remain operational throughout construction as noted above. Recreational marine operations would be impacted somewhat due to the work at Spans 5, 6, and 7.

6. Utility Impacts

TBD

7. Right of Way Impacts

No permanent right of way impacts are required for this alternative, as the existing bridge is located within the State of Connecticut Right of Way. Temporary construction easements may be required for the contractor's staging area and access to the bridge.

8. Environmental Impacts

Encasement of the existing piers will present environmental impacts that will require mitigation measures. In addition, any work by the contractor on the structural steel would require proper measures to prevent lead paint from entering the river or upland areas. The contractor would be required to follow Best Management Practices.

Depending on the contractor's use of falsework within the river, work associated with the removal and installation of trusses at Spans 5, 6, and 7 may produce temporary environmental impacts.

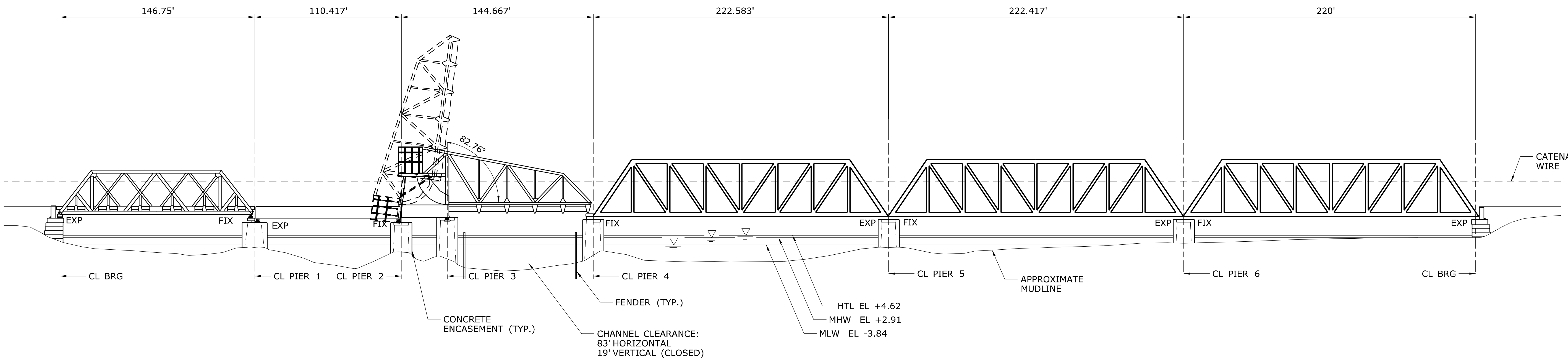
9. Historical Impacts

The replacement of the existing trusses of Spans 5, 6 and 7 will adversely affect the structure from a historical perspective. The replacement trusses will be appear similar in nature and partially mitigate the loss of the original span appearance. In addition, encasing the existing piers in concrete to improve seismic performance will also alter the historic nature of the bridge. Coordination with SHPO will be required, and proper documentation of spans to be replaced will need to be performed.

10. Hydraulic Impacts

Encasing of the existing piers will result in an increase of flow obstruction, and will likely negatively affect hydraulic performance at the bridge. A full hydraulic and scour analysis is required to determine the actual impacts associated with this alternatives.

During construction, cofferdams necessary for retrofitting the piers will be present in the waterway. A temporary facilities hydraulic analysis will be required to determine if there will be adverse affects resulting from the temporary hydraulic conditions anticipated during construction.



ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVE IIIa - ELEVATION	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-301A

F.) Alternative IIIb – Complete Superstructure Replacement

1. Description

This alternative consists of replacing all superstructure spans truss structures, performing repairs to and modifying the substructure, and replacing all of the mechanical and electrical components with new. If the movable span is replaced with a vertical lift, new piers will be required to support the lift towers and span. Span 1 will be replaced in kind with a new truss. The new spans at Spans 5, 6, and 7 would also be similar to existing, with the exception that the truss members would be rolled steel sections and would be connected using bolted gusset plates. The floor system for Spans 1, 5, 6, and 7 would be a floorbeam and stringer configuration similar to existing. The signals and communication system will be replaced, as will the catenary and all track on the bridge.

Several movable span options are available with this alternative; the configuration of the movable span will dictate the structure type for Spans 2, 3 and 4. Available options for the movable span include:

- Rolling Bascule (Scherzer Rolling Lift)
- Heel Trunnion Bascule
- Vertical Lift

A Simple Trunnion Bascule is not practical at this location due to the need for a counterweight pit, while a Swing Bridge is not practical due to limited horizontal clearance with the Moses Wheeler Bridge.

With both bascule alternatives, the structure types for Spans 2, 3 and 4 would remain essentially the same as existing, with Spans 3 and 4 adjusted to accommodate the particular movable span type.

The vertical lift alternative would allow for flexibility of structure type for Span 2, as vertical clearance above the track at this span will not be required as it is for the bascule counterweight options. In addition Span 3 would be eliminated, as new piers to support the lift span towers will be constructed and the existing Pier 3 will be eliminated.

However, the existing pier layout naturally favors reusing the existing movable span type of a Rolling Bascule in terms of cost and constructability when considering a superstructure replacement. As such, only the Rolling Bascule was progressed and developed as a movable span type for this alternative.

The existing masonry piers to remain will be retrofitted with concrete encasement to improve seismic performance.

The existing high towers will be replaced with three new monotube towers independent of the bridge structure.

2. Sequencing and Duration

The majority of the work contemplated with this alternative would be completed during a two track outage of the railroad, whereby each of the two independent truss systems are replaced and rehabilitated separately. Taking tracks out of service is necessary to replace the existing trusses with new. The duration of the two track outage depends highly on the method of construction chosen for the replacement of trusses at Spans 5, 6, and 7. See Item 3 – Constructability below.

<u>Stage 1</u>

Stage 1 will consist of work on the northerly truss system, with long term track outages to Tracks 1 and 3. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

After the tracks are taken out of service, work would begin on removing and replacing Spans 1 through 7. The northerly components of the new movable span will be constructed at this time. A new Operator's House will also be constructed during this stage to the north of the structure.

The duration of Stage 1 will last approximately 20 to 30 months. The duration of this stage is driven primarily by the construction of new substructures for the movable span structure and associated support elements. The duration of construction will also depend somewhat on the method the contractor uses to install the new trusses at Spans 5, 6, and 7. If these trusses are constructed off-site and jacked into position, the duration for this portion will be considerably less than if the trusses are constructed in place using falsework.

Stage 2

Stage 2 will consist of the rehabilitation of the southerly truss system, with long term track outages to Tracks 2 and 4. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

As with Stage 1, work would begin on removing and replacing Spans 1 through 7 after the tracks are taken out of service along with construction of the new movable span components.

The duration of Stage 2 will last approximately 20 to 30 months. The duration this stage is also driven primarily by the construction of new substructures for the movable span structure and associated support elements. The duration of construction will also depend somewhat on the method the contractor uses to install the new trusses at Spans 5, 6, and 7. If these trusses are constructed off-site and jacked into position, the duration for this portion will be considerably less than if the trusses are constructed in place using falsework.

Stage Independent Work

Other work associated with this alternative, such as repairs to the Operator's House, associated framing and the substructure, can occur concurrent with the above Stage Construction work.

This section will be expanded later upon acceptance of the two track outage railroad sequencing scenario.

3. Constructability

At Spans 1, 2, 3, and 4 – Bascule Option (Rolling Lift and Heel Trunnion):

Span 1 would be removed using either falsework below the existing trusses to facilitate disassembly of the existing truss and construction of the new truss, or through the use of large crane picks.

Installation of the new truss would proceed in a similar fashion. If installed using a crane, the bridge could be constructed off site or assembled nearby either upstream or downstream.

The Span 2 girders would be removed and installed using a crane staged from barge(s) below. The Span 3 superstructure could be removed and installed using either a barge from the water below, or from track mounted cranes from the track level. The rack structure would be removed and constructed in a similar fashion.

The movable Span 4 trusses would be removed and installed in a similar fashion as Span 1.

At Spans 1, 2, 3, and 4 – Vertical Lift Option:

Span 1 would be removed using either using falsework below the existing trusses to facilitate disassembly of the existing truss and construction of the new truss, or through the use of large crane picks. Spans 2 and 3 superstructure would be removed using a crane staged from barge(s) below. The movable Span 4 trusses would be removed and installed in a similar fashion as Span 1.

With the superstructures removed, a cofferdam around existing Piers 2, 3, and 4 would be installed to facilitate their removal to below the river bed. Demolition of the piers would proceed in stages taking into account the staging of the railroad operations above. Two new piers would then be installed to support Span 2, the new lift span (Span 3/4) and towers, and Span 5.

Installation of the cofferdams and access to the piers would be accomplished from barges in the water.

At Spans 5, 6 and 7:

The replacement of the existing trusses could proceed using several methods. The most basic construction option is to construct falsework below the existing trusses to facilitate disassembly of the existing truss and construction of the new truss. Falsework supports would extend to the riverbed, and would remain in place until all three trusses have been removed and new trusses installed.

Alternatively, the trusses may be able to be floated in to position on barges, and jacked into position from the barges. For this scenario, the trusses would be constructed either off site, assembled nearby either upstream or downstream of the bridge, or a combination of both. The widening of the Moses Wheeler Bridge reduces the clearance between the two bridges; however there will be adequate room for a new truss to be maneuvered between the two bridges.

The operation would consist of a barge being floated into place under the existing truss. Once the barge was anchored to the riverbed and tracks cut between spans, a system of falsework and hydraulic jacks would be used to raise the bridge off of its bearings. When adequate vertical clearance is obtained, the truss would be translated laterally until clear of the piers. With the existing truss removed, any preparation required for the new trusses would occur, such as bridge seat repairs and bearing installation. With the bridge prepared, the new truss would then be floated into position on a barge and lowered onto the existing substructure, using jacks for final positioning. The contractor would need to account for tidal fluctuations during the removal and installation operations.

If the new trusses are constructed off site, the northerly trusses would be floated into position under I-95 and through the opened bascule of the Devon Bridge via the navigation channel. This operation becomes more complicated for installation when transporting the southerly trusses, as the final pier sizes for the Moses Wheeler Bridge have not been established. There will be ample horizontal clearance between I-95 piers at the navigation channel; however the fender system would need to be removed first. Further investigation is required to determine if there is adequate clearance for the trusses to be maneuvered under other spans of the Moses Wheeler Bridge.

The southerly trusses could also be constructed on site, on the east bank in the area between the Moses Wheeler Bridge and the Devon Bridge, and then launched west into the Housatonic River and into place.

Removal and installation of Span 7 trusses cannot be accomplished using a barge due to the shallow water depths under that span, and a combination of falsework and/or cranes would be required to disassemble existing and install new trusses.

4. Railroad Operational Impacts

Impacts to railroad operations resulting from work proposed in this alternative would be substantial, and will result in impacts to operations between CP 255 and CP 261 at a minimum. A minimum of four (4) miles of two track service is necessary unless a new series of interlockings are added between CP 255 and CP 261. This two track service will affect the operational flexibility of the railroad. By only allowing for one track in each direction, peak hour service, especially as relates to express and local trains, would be affected.

Short term track outages may be utilized during off peak times, typically with only one track removed from service at a time. Trains from the Waterbury Branch complicate track outages on Track 3, and significantly limits the effective window of working time. Scheduling for a longer term track outage may be required for Track 3 to provide the contractor with realistic working windows.

Temporary platforms would be required at one rail station at a minimum during one of the stages of construction.

Refer to Section 5.B of this report for additional discussion.

5. Marine Operational Impacts

Impacts to marine operations will depend on the bridge demolition and installation alternative chosen. If temporary falsework is used to provide support, the waterway under these spans would be unusable to marine traffic.

Overall, the impacts to marine operations would be minimal. The navigation channel would remain operational for the majority of the construction, except during installation of cofferdams around substructure units, and during removal, installation, and testing of the lift span. At these times, the navigation channel would not be accessible.

6. Utility Impacts

TBD

7. Right of Way Impacts

No permanent right of way impacts are required for this alternative, as the existing bridge is located within the State of Connecticut Right of Way. Temporary construction easements may be required for the contractor's staging area and access to the bridge.

8. Environmental Impacts

Encasement of the existing piers will present environmental impacts that will require mitigation measures. In addition, any work by the contractor on the structural steel would require proper measures to prevent lead paint from entering the river or upland areas. The contractor would be required to follow Best Management Practices.

Depending on the contractor's use of falsework within the river, work associated with the removal and installation of the new superstructure may produce temporary environmental impacts.

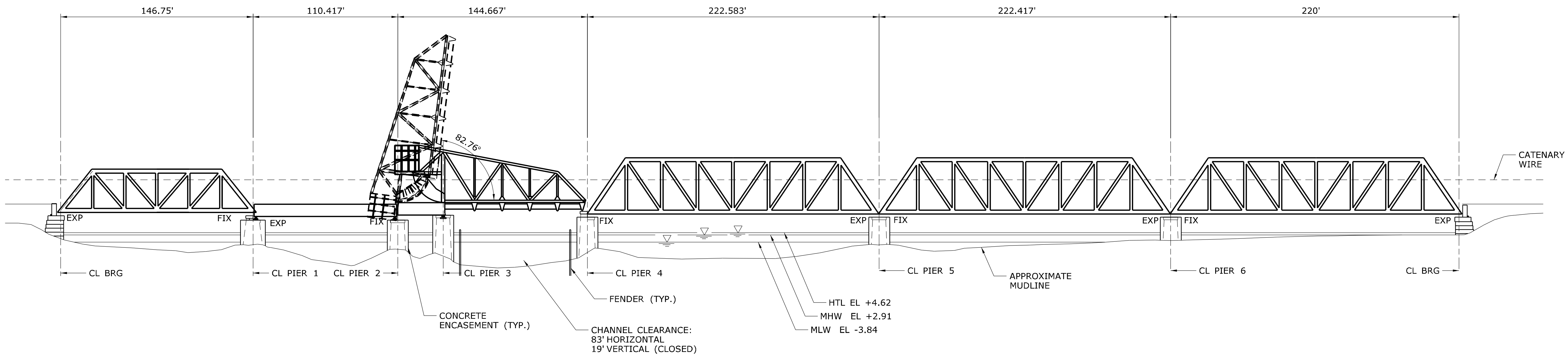
9. Historical Impacts

The replacement of the existing trusses of Spans 5, 6 and 7 will adversely affect the structure from a historical perspective. The replacement trusses will be appear similar in nature and partially mitigate the loss of the original span appearance. In addition, encasing the existing piers in concrete to improve seismic performance will also alter the historic nature of the bridge. Coordination with SHPO will be required, and proper documentation of spans to be replaced will need to be performed.

10. Hydraulic Impacts

Encasing of the existing piers will result in an increase of flow obstruction, and will likely negatively affect hydraulic performance at the bridge. A full hydraulic and scour analysis is required to determine the actual impacts associated with this alternatives.

During construction, cofferdams necessary for retrofitting the piers will be present in the waterway. A temporary facilities hydraulic analysis will be required to determine if there will be adverse affects resulting from the temporary hydraulic conditions anticipated during construction.



ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVE IIIb - ELEVATION	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-301B

G.) Alternatives IVa & IVb – Full Replacement

1. Description

These two alternatives involve the complete replacement of the existing bridge on the existing alignment.

Alternative IVa

This alternative replaces the existing structure with five spans of approximately 217 feet, 200 feet, 217 feet, 217 feet, and 188 feet. The spans will consist of through trusses similar to that of the existing bridge in form. However the truss members will consist of rolled steel shapes connected with bolted gusset plates. The floor system would be a floorbeam and stringer configuration similar to existing.

Alternative IVb

This alternative replaces the existing structure with eight spans of approximately 114 feet, 114 feet, 200 feet, 126 feet, 126 feet, 126 feet, 126 feet, and 126 feet at the Devon Bridge. The spans will consist of deck girders similar to that of the existing Span 2, with the movable span consisting of a truss due to the long span of the vertical lift.

For both alternatives, the substructures will be constructed of reinforced concrete abutments and piers supported by driven/drilled shafts. The deep foundations would be socketed into bedrock.

Due to the long span at the navigation channel, the only practical configuration for the movable span is a vertical lift supported by a through truss. Similar to existing, the lift span will consist of two independent structures, with separate supports, machinery, controls, etc., to allow for redundancy. Each structure will support two tracks. The signals and communication system will be replaced, as will the catenary and all track on the bridge.

The existing high towers will be replaced with three new monotube towers independent of the bridge structure.

2. Sequencing and Duration

The majority of the work contemplated with these alternatives will be completed during one of two track outages of the railroad, whereby each of the two independent truss systems are replaced and rehabilitated separately. The sequence of construction is the same for both Alternatives IVa and IVb.

Stage 1

Stage 1 will consist of work on the northerly structural system, with long term track outages to Tracks 1 and 3. Refer to Section V.B – Operations During Construction for specific sequencing of the track outages.

Prior to taking the tracks out of service, temporary construction platforms would be installed on the north side of the bridge to facilitate installation construction access across the river. On the east bank, this access would be provided from the boat launch area. On the west bank, this access would be provided from the parking area between the Moses Wheeler Bridge and the Devon Bridge. The navigation channel would remain clear.

After the tracks are taken out of service, the existing superstructure will be removed. With the superstructure out of the way, construction of the new substructure will begin with the installation of the drilled shafts for the piers at north half of the structure as well as construction of the north half of the abutments. With the drilled shafts installed, a pier cap/bridge seat connecting to the drilled shafts (or columns extending from the shafts) will be installed at each pier. The top portion of each of the existing piers will then be removed to allow for clearance while the new superstructures are installed; however, the majority of the pier will remain until Stage 2. A cofferdam around each pier will be installed north of the bridge centerline while the superstructure is removed, allowing for clearance for installation of the cofferdam.

The northerly components of the vertical lift span towers for the new movable span will be constructed at this time with the new movable span. A new Operator's House will also be constructed during this stage to the north of the structure. The existing Operator's House will remain in service during this stage. The new superstructures will be installed on the new piers at this time.

This stage will conclude when Tracks 1 and 3 are placed back in service.

The duration of Stage 1 will last approximately 2 years.

Stage 2

Stage 2 will consist of work on the southerly structural system, with long term track outages to Tracks 2 and 4. The temporary construction access would be extended south as required for foundation installation access.

Prior to taking Tracks 2 and 4 out of service, the new Operator's House will be commissioned and begin to control operation of the movable spans. Control of the existing span will need to be transferred to the new Operator's House as well. After the tracks are taken out of service, the existing southerly superstructure will be removed, along with the Operator's House. As with Stage 1, the installation of the drilled shafts will occur next. The pier cap/bridge seat would be completed next, tying the piers for the two stages together. At this time, the remainder of the cofferdam will be installed around the existing piers, and they will be removed in their entirety, including those portions below the northerly structures. Then the superstructures would be installed atop the new piers, along with the southerly components of the vertical lift span towers and span. The cofferdam would then be cut off at the mudline and removed.

The duration of Stage 2 will last approximately 2 years.

3. Constructability

The removal of the existing bridge superstructure, and subsequent installation of the new superstructures, could proceed using several methods. The most basic construction option is to construct falsework below the existing trusses to facilitate disassembly of the existing truss and construction of the new truss. Falsework supports would extend to the riverbed, and would remain in place until all three trusses have been removed and new trusses installed. However, the differences in span configuration between the existing and new bridges will reduce the efficiency of this method.

Alternatively, the contractor may mount the falsework on barges for Spans 2, 3, 4, 5, and 6. The operation would consist of a barge being floated into place under the existing truss. Once the barge was anchored to the riverbed and tracks cut between spans, a system of falsework and hydraulic jacks would be used to raise the bridge off of its bearings. When adequate vertical clearance is obtained, the truss would be translated laterally via barge until clear of the piers. The contractor would need to account for tidal fluctuations during the removal and installation operations.

Removal and installation of Span 1 and 7 trusses cannot be accomplished using a barge due to the shallow water depths under that span, and a combination of falsework and/or cranes would be required to disassemble existing and install new trusses.

For Alternative IVa, installation of the new superstructure would be installed in a similar fashion from a barge. For this scenario, the trusses would be constructed either off site, assembled nearby either upstream or downstream of the bridge, or a combination of both. The widening of the Moses Wheeler Bridge reduces the clearance between the two bridges; however there appears to be adequate room for a new truss to be maneuvered between the two bridges.

If the new trusses are constructed off site, the trusses would be floated into position under I-95 and through the opened bascule of the Devon Bridge via the navigation channel. The temporary work platform would need to be partially dismantled to allow the barges to be located under the existing trusses. This operation becomes slightly more complicated for installation when transporting the southerly trusses due to the Moses Wheeler Bridge piers. It appears there will be adequate horizontal clearance between I-95 piers at the navigation channel; however the fender system would need to be removed first.

For Alternative IVb, the lighter weight of the steel girders would allow for cranes to pick the steel while accessing the bridge from the temporary platform. This would eliminate the need to use barges to float the new trusses into place, simplifying construction.

The southerly trusses could also be constructed on site, on the east bank in the area between the Moses Wheeler Bridge and the Devon Bridge, and then launched west into the Housatonic River and into place.

<u>Foundation Installation Considerations</u>

The construction of an entirely new structure on the same alignment as existing, coupled with the need to maintain at least half of the existing/new structure in service at all times, presents constructability issues.

The preliminary layout of the new piers for both alternatives generally do not align with the existing piers of the Devon Bridge. This allows for the existing piers to remain in place while the new piers are installed, affording flexibility with the construction sequencing. In addition, CTDOT has encountered complications with pier stability during recent construction projects involving removal of masonry piers in stages. Based on this experience, it is preferable to decommission the piers in their entirety as opposed to removing portions of them in stages.

The above staging accounts for installation of the drilled shafts during both Stages 1 and 2. Further analysis is required to determine the actual number and size of the drilled shafts. However, if three shafts per pier are required, the middle and northerly shafts of each pier would be installed during Stage 1 with the middle shaft offset from the center to allow for clearance from the active southerly structure.

It may also be possible to install all of the drilled shafts at once, eliminating duplicate mobilization costs. This could be accomplished by installing the middle and northerly shafts as noted above, with the southerly shafts having been installed prior to Stage 1 immediately to the south of the existing bridge. Then during Stage 2, the only substructure components to construct would be completion of the pier caps (connecting the southerly shaft with the middle and northerly assembly completed during Stage 1) and abutments. This sequence would decrease the duration of Stage 2 considerably.

4. <u>Railroad Operational Impacts</u>

This section will be expanded later upon acceptance of the two track outage railroad sequencing scenario.

Impacts to railroad operations resulting from work proposed in this alternative would be substantial, and will result in impacts to operations between CP 255 and CP 261 at a minimum. A minimum of four (4) miles of two track service is necessary unless a new series of interlockings are added between CP 255 and CP 261. This two track service will affect the operational flexibility of the railroad during construction. By only allowing for one train in each direction, peak hour service, especially as relates to express and local trains, would be affected.

Short term track outages may be utilized during off peak times, typically with only one track removed from service at a time. Trains from the Waterbury Branch complicate track outages on Track 3, and significantly limits the effective window of working time. Scheduling for a longer term track outage may be required for Track 3 to provide the contractor with realistic working windows.

Temporary platforms would be required at one rail station at a minimum during one of the stages of construction.

Refer to Section 5.B of this report for additional discussion.

5. Marine Operational Impacts

Impacts to marine operations will depend on the bridge demolition and installation alternative chosen. If temporary falsework is used to provide support, the waterway under these spans would be unusable to marine traffic.

Overall, the impacts to marine operations (navigation) would be minimal. The navigation channel would remain operational for the majority of the construction, except during installation of cofferdams around substructure units, and during removal, installation, and testing of the lift span. At these times, the navigation channel would not be accessible.

6. Utility Impacts

TBD

7. Right of Way Impacts

No permanent right of way impacts are required for this alternative, as the existing bridge is located within the State of Connecticut Right of Way. Temporary construction easements may be required for the contractor's staging area and access to the bridge.

8. Environmental Impacts

Significant environmental impacts are associated with this alternative. Three new piers will be installed in the waterway, impacting the river bed at these locations. However, these impacts will be mitigated somewhat by the removal of the six existing bridge piers. Portions of the existing and new piers will exist simultaneously in the river bed during construction, temporarily reducing the waterway area.

Aside from normal environmental construction measures, any work by the contractor would require proper measures to prevent debris from entering the river or upland areas. The contractor would be required to follow Best Management Practices.

Depending on the contractor's use of falsework within the river, work associated with the removal and installation operations may produce temporary environmental impacts.

9. Historical Impacts

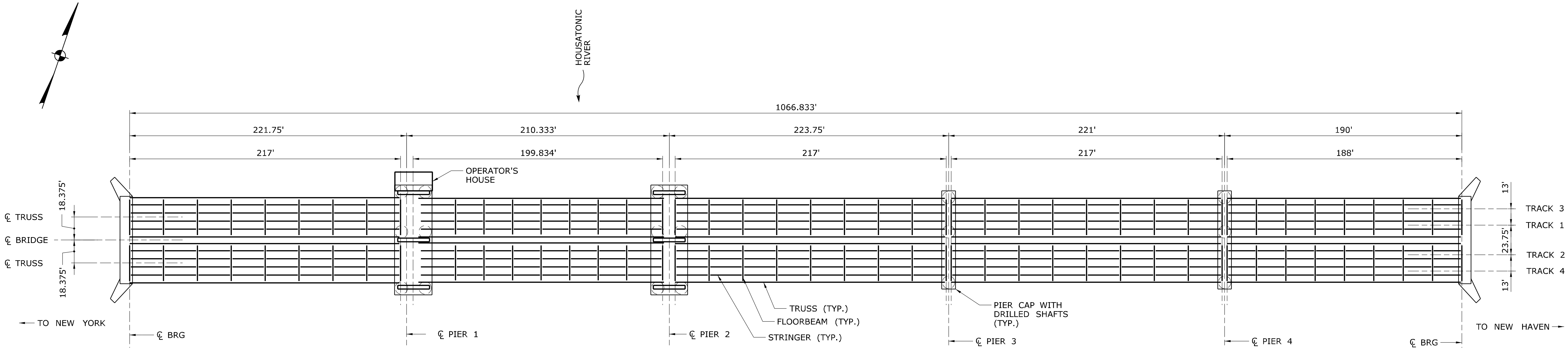
Both Alternative IVa and IVb will have adverse historical impacts, as the existing bridge will be removed in its entirety. Coordination with SHPO will be required, and proper documentation of spans to be replaced will need to be performed.

10. Hydraulic Impacts

Any construction that adds or removes obstructions from the river will affect flow characteristics, including up- and downstream flood elevations, flow velocities, and predicted scour depths. A full hydraulic and scour analysis is required to determine the actual impacts associated with these two alternatives. However, removal of the existing piers and installation of the new piers of Alternative IVa will result in a net decrease of obstructions within the waterway. This will likely improve flow characteristics of the river under the bridge, and should not result in adverse upstream hydraulic effects. in the waterway area.

Because there are more piers required in the waterway for Alternative IVb, further analysis is required before making a judgment as to hydraulic effects of this alternative. In general, the diameter of the drilled shafts will need to be minimized in order to keep the area of obstructions in the river to a minimum.

During construction, both existing and proposed piers, as well as temporary construction access, will be present in the waterway. The flow area will further be constricted by the cofferdams required for removal of the existing piers. A temporary facilities hydraulic analysis will be required to determine if there will be adverse affects resulting from the temporary hydraulic conditions anticipated during construction.



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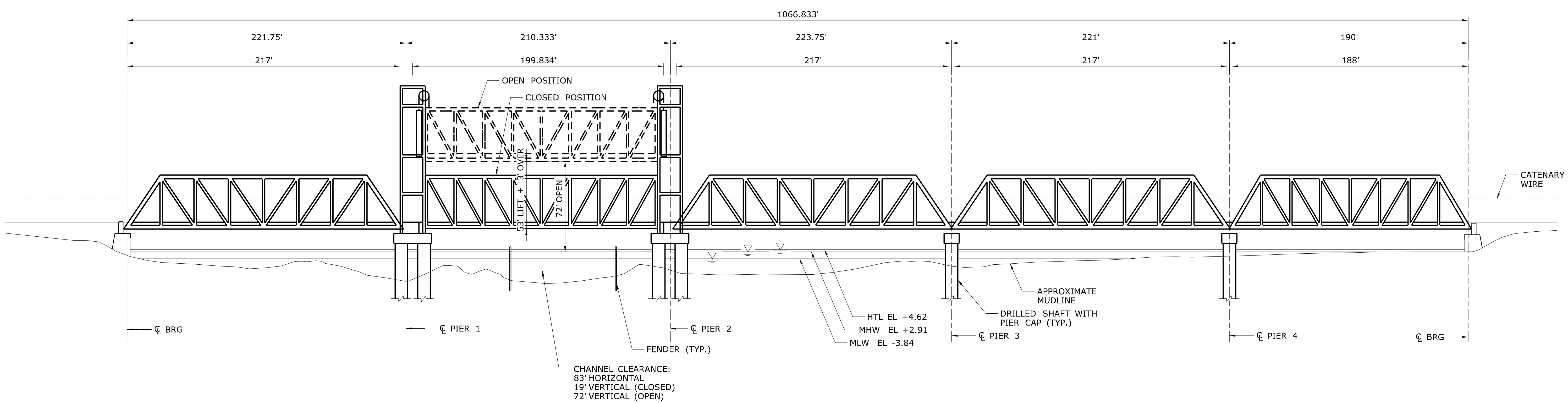
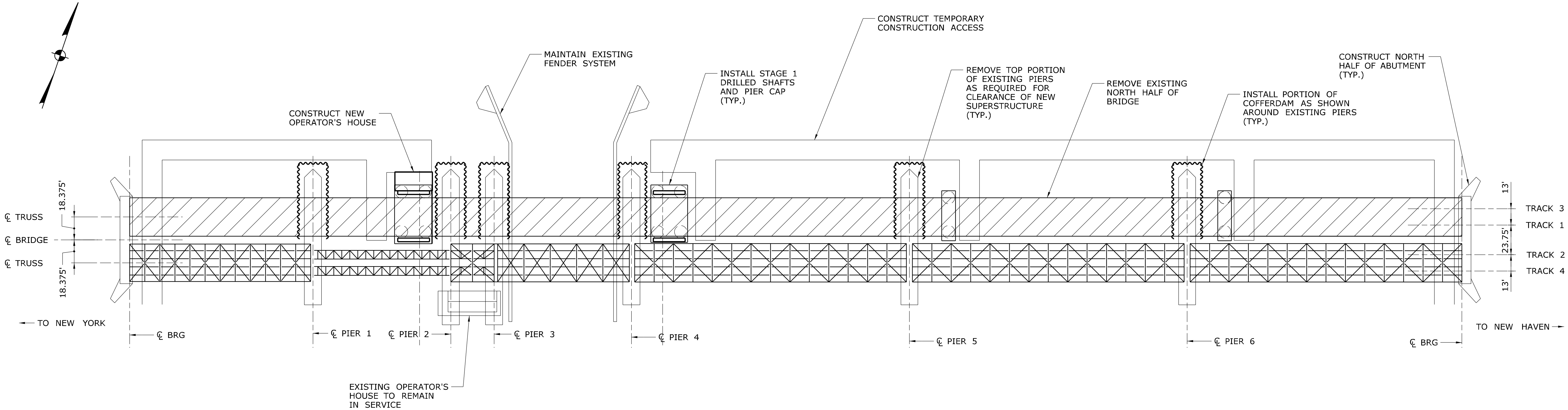


ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVE IVa - VERTICAL LIFT	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-401A



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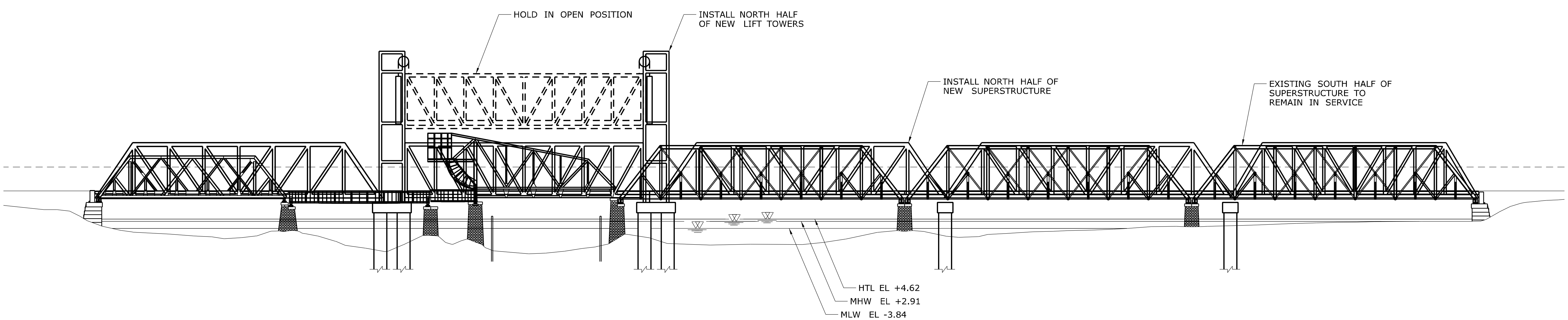
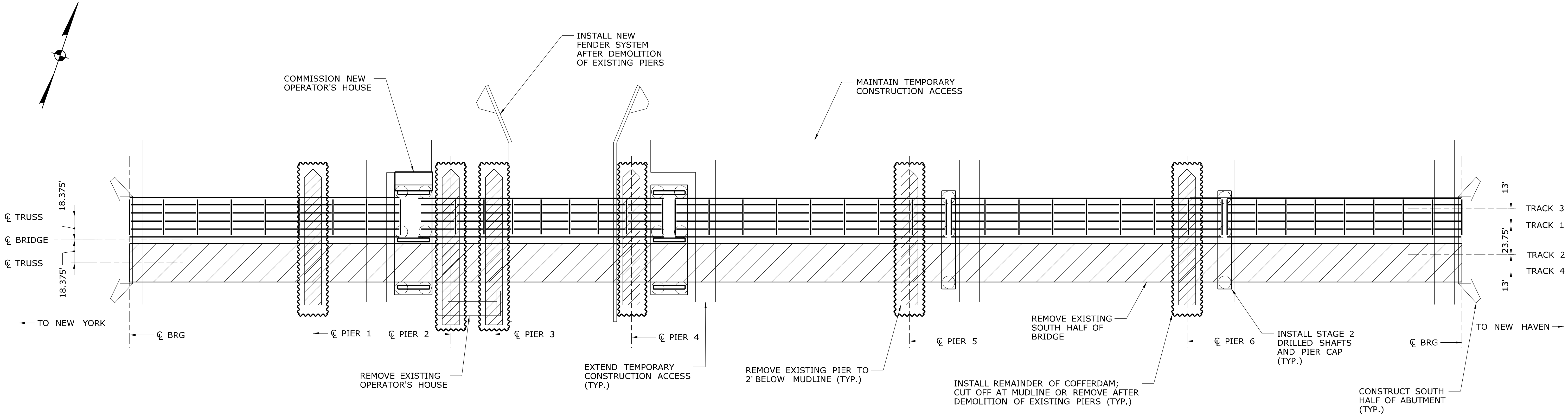


STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	ALTERNATIVE IVa - STAGE 1	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE:
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	1" = 80'
		FIGURE: S-403A



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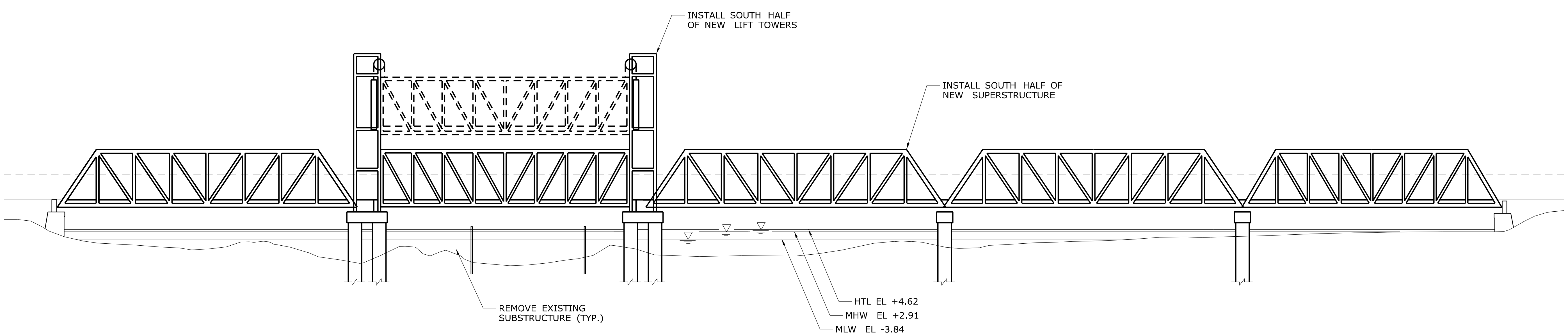
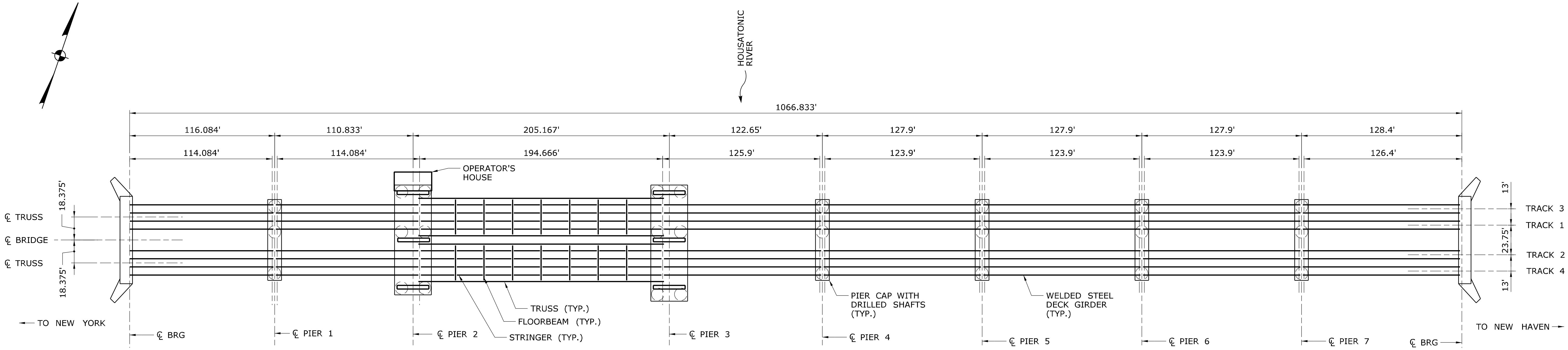


STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVEa IV - STAGE 2	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-404A



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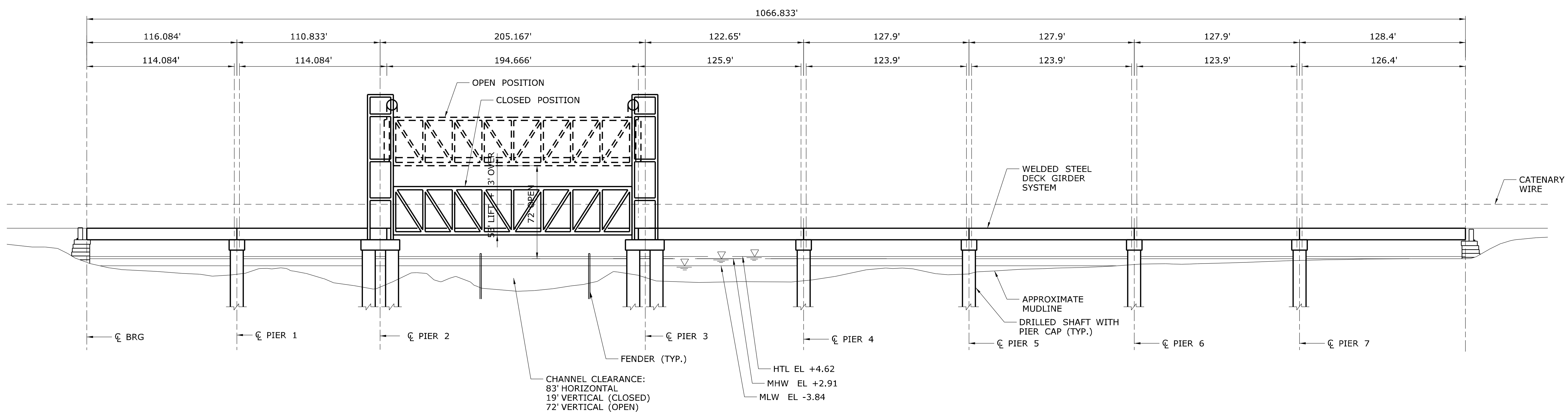
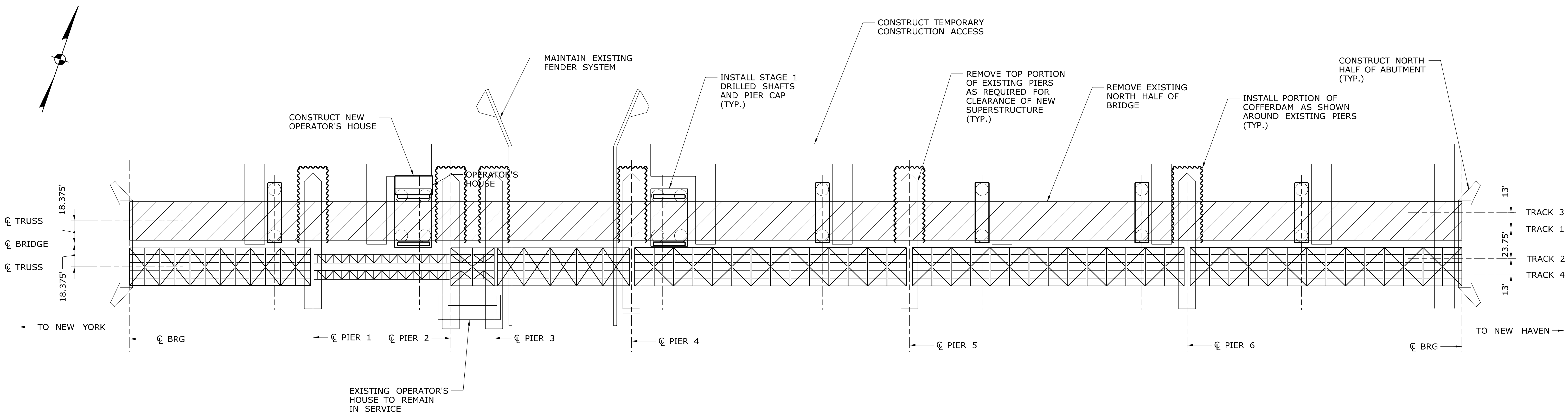


ELEVATION

STATE PROJECT NO.: 301-099	DRAWING TITLE: ALTERNATIVE IVb - VERTICAL LIFT	DATE: JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80' FIGURE: S-401B



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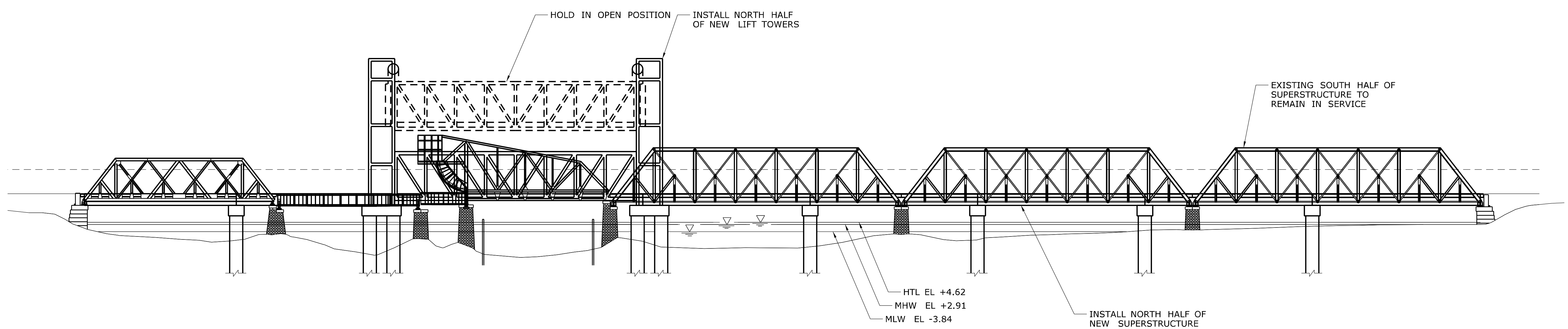
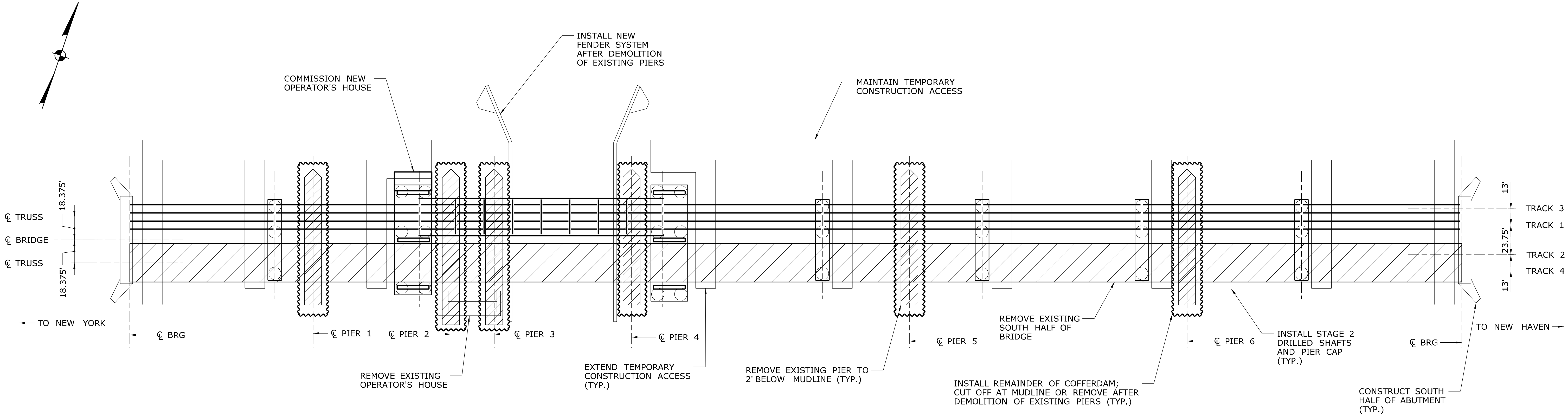


STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	ALTERNATIVE IVb - STAGE 1	JUNE 2010
CITY/TOWN:	PROJECT:	SCALE: 1" = 80'
MILFORD/STRATFORD	ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	FIGURE: S-403B



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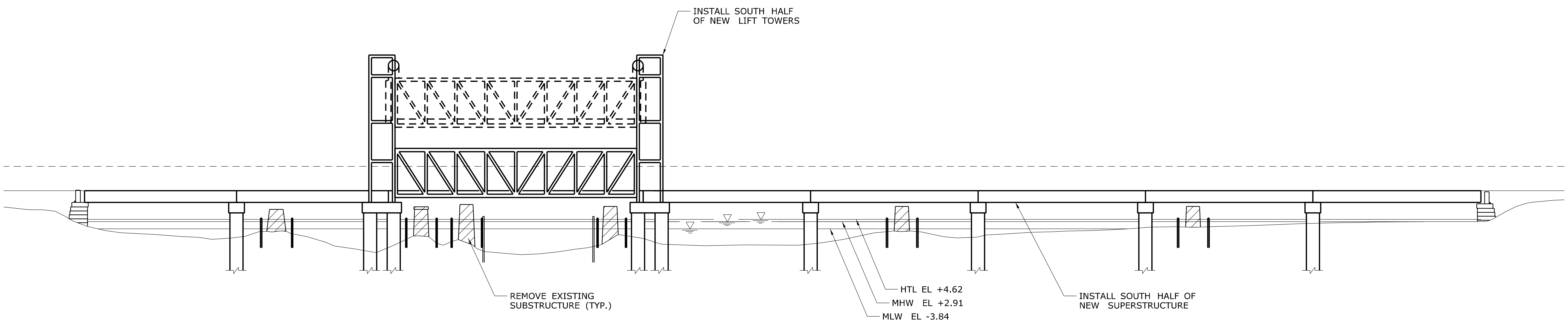

ELEVATION

STATE PROJECT NO.:	DRAWING TITLE:	DATE:
301-099	ALTERNATIVE IVb - STAGE 2	JUNE 2010
CITY/TOWN: MILFORD/STRATFORD	PROJECT: ENGINEERING, FEASIBILITY AND ECONOMIC ANALYSIS STUDY OF METRO-NORTH RAILROAD BRIDGE OVER THE HOUSATONIC RIVER (DEVON BRIDGE)	SCALE: 1" = 80'
		FIGURE: S-404B

XII. Alternatives Cost Analysis

A.) Preliminary Cost Estimates

Capital cost estimates were developed for each of the six alternatives, with costs developed in year 2010 dollars based on unit costs including labor, material, and equipment expenses for each item. Miscellaneous Items (Mobilization, Minor Items, etc.) and construction cost contingency were added to this cost on a percentage basis of 40% and 20% respectively, as directed by CTDOT staff. Costs for Signals & Communication, Catenary, and Track & Rail work were developed by CTDOT staff and added to this subtotal to develop a Total Construction Cost for each alternative.

Additional items were added on a percentage basis as directed by CTDOT staff for Force Account work (40%), Engineering (15%), Incidental Expenses (20%). Finally, an overall project contingency of 15% was added to develop the total project cost. The total project cost was then projected out to the anticipated midpoint of construction at a rate of 6% per year to develop the anticipated future project cost.

A summary of the results of the cost analysis are presented in the following table:

Preliminary Cost Estimate*	
Alternative I	$3,000,000**
Alternative II	$280,000,000
Alternative IIIa	$580,000,000
Alternative IIIb	$660,000,000
Alternative IVa	$840,000,000
Alternative IVb	$790,000,000

*Force Account, Catenary, Signals & Communications, Traction Power, & Station Access costs provided by CTDOT staff
**Assumes long term rehabilitation to follow within 5-7 years

A detailed breakdown of the preliminary cost estimates for each alternative is presented in the following pages.

Alternative I – Short Term Repairs

Estimate of Short Term Repairs (5-7 year)

Stringer Repairs - MNR Labor

	# of Locations	# Days per Repair	Total Days	Rate/day	Total
TF only	84	4	336	$ 4,000	$ 1,344,000
BF only	17	3	51	$ 4,000	$ 204,000
Both Flanges	8	5	40	$ 4,000	$ 160,000
					$ 1,708,000

Miscellaneous Other Repairs (FB, Rivets, etc.) - MNR Labor

	# of Locations	# Days per Repair	Total Days	Rate/day	Total
Miscellaneous	21	4	84	$ 4,000	$ 336,000
					$ 336,000

Materials & Equipment

Subtotal MNR Labor	$ 2,044,000
15% Materials	$ 306,600
10% Equipment	$ 204,400
20% Incedental	$ 408,800
Total	$ 2,963,800

Estimated Total

say: $ 3,000,000

Notes:
1. Repairs to be performed by MNR forces
2. Assumes comprehensive rehabilitation/replacement project to begin approximately 7 years after implementation of repairs.
4. Cost of MNR Forces assumed at $4,000 per day, per L. Laber, 4/26/10
5. Material and Equipment Rates per L. Laber, 4/26/10

Alternative II – Rehabilitation

Alternative II - Rehabilitation

Base Year: 2010
Midpoint of Construction: 2019

Preliminary Construction Cost Estimate and Total Project Cost

Item No.	Item	Unit	Quantity		Unit Price		Price
Structure Items							
	Steel Repairs	LS	LS	$	9,750,000	$	9,750,000
	Pier Repointing/Repair	SY	4950	$	300	$	1,485,000
	Scour Monitoring Equipment	LS	LS	$	1,000,000	$	1,000,000
	Class 3 Containment	SF	615000	$	20	$	12,300,000
	Cleaning and Painting Structural Steel	SF	615000	$	20	$	12,300,000
	Operator's House	LS	LS	$	-	$	-
	Mechanical & Electrical	LS	LS	$	3,000,000	$	5,000,000
	Fender System	LS	LS	$	500,000	$	1,500,000
	Temporary Construction Access	LS	LS	$	-	$	-
Total Structure Items (2010 Dollars)						*$*	*43,335,000*
Other Items							
	High Towers	EA	3	$	2,000,000	$	6,000,000
Total Other Items (2010 Dollars)						*$*	*6,000,000*
Subtotal (Structure + Other Items)						*$*	*49,335,000*
Miscellaneous Items (Mobilization, Minor Items, etc.) @ 40%						*$*	*19,734,000*
Contingency @ 20%						*$*	*9,867,000*
Signals & Communication						*$*	*2,000,000*
Catenary						*$*	*-*
Track & Rail						*$*	*4,500,000*
Total Construction Cost						*$*	*85,436,000*
Force Account / RR Protection Services @ 40% of Construction Cost						*$*	*34,174,400*
Engineering Fee @ 15% of Construction Cost						*$*	*12,815,400*
Incidental Expenses @ 20% of Construction Cost						*$*	*17,087,200*
Contingency @ 15% of Construction Cost						*$*	*12,815,400*
					Subtotal (2010 Dollars)	*$*	*162,328,400*
			Assumed Annual Inflation: 6.00%			*$*	*111,922,016*
					Total Project Cost at Midpoint of Construction (2019 Dollars)	***$***	***274,250,416***
					Say:	***$***	***280,000,000* ****

**Cost Adjusted to inflation 6% annually to the midpoint of construction*

Notes:

1. Percentages for Miscellaneous Items, Contingencies, Force Account, Engineering Fee, Incidental Expenses are as directed by CTDOT staff.
2. Costs for Signals & Communication, Catenary, Station Access, and Track & Rail work developed by CTDOT staff.

Alternative IIIa – Partial Superstructure Replacement

Alternative IIIa - Partial Superstructure Replacement

Base Year: 2010

Midpoint of Construction: 2020

Preliminary Construction Cost Estimate and Total Project Cost

Item No.	Item	Unit	Quantity	Unit Price	Price
Structure Items					
	Removal of Superstructure	LS	LS	$ 6,500,000	$ 6,500,000
	Cofferdam and Dewatering	LF	1800	$ 1,500	$ 2,700,000
	Pier Repairs and Retrofit	LS	LS	$ 15,000,000	$ 15,000,000
	Scour Monitoring Equipment	LS	LS	$ 1,000,000	$ 1,000,000
	Structural Steel (Spans 5-7)	CWT	58800	$ 650	$ 38,220,000
	Steel Repairs	LS	LS	$ 6,000,000	$ 6,000,000
	Class 3 Containment	SF	240000	$ 20	$ 4,800,000
	Cleaning and Painting Structural Steel (Spans 1-4)	SF	240000	$ 20	$ 4,800,000
	Operator's House	LS	LS	$ 750,000	$ 750,000
	Mechanical & Electrical	LS	LS	$ 5,000,000	$ 5,000,000
	Fender System	LS	LS	$ 500,000	$ 1,500,000
	Temporary Construction Access	LS	LS	$ -	$ -
Total Structure Items (2010 Dollars)					*$ 86,270,000*
Other Items					
	High Towers	EA	3	$ 2,000,000	$ 6,000,000
Total Other Items (2010 Dollars)					*$ 6,000,000*
Subtotal (Structure + Other Items)					*$ 92,270,000*
Miscellaneous Items (Mobilization, Minor Items, etc.) @ 40%					*$ 36,908,000*
Contingency @ 20%					*$ 18,454,000*
Signals & Communication					*$ 4,000,000*
Catenary					*$ 8,000,000*
Track & Rail					*$ 9,000,000*
Total Construction Cost					*$ 168,632,000*
Force Account / RR Protection Services @ 40% of Construction Cost					*$ 67,452,800*
Engineering Fee @ 15% of Construction Cost					*$ 25,294,800*
Incidental Expenses @ 20% of Construction Cost					*$ 33,726,400*
Contingency @ 15% of Construction Cost					*$ 25,294,800*
				Subtotal (2010 Dollars)	*$ 320,400,800*
			Assumed Annual Inflation: 6.00%		*$ 253,388,235*
				Total Project Cost at Midpoint of Construction (2020 Dollars)	***$ 573,789,035***
				Say:	***$ 580,000,000*** *

**Cost Adjusted to inflation 6% annually to the midpoint of construction*

Notes:

1. Percentages for Miscellaneous Items, Contingencies, Force Account, Engineering Fee, Incidental Expenses are as directed by CTDOT staff.
2. Costs for Signals & Communication, Catenary, Station Access, and Track & Rail work developed by CTDOT staff.

Alternative IIIb – Full Superstructure Replacement

Alternative IIIb - Full Superstructure Replacement

Base Year: 2010

Midpoint of Construction: 2020

Preliminary Construction Cost Estimate and Total Project Cost

Item No.	Item	Unit	Quantity		Unit Price		Price
Structure Items							
	Removal of Superstructure	LS	LS	$	10,500,000	$	10,500,000
	Cofferdam and Dewatering	LF	1800	$	1,500	$	2,700,000
	Pier Repairs and Retrofit	LS	LS	$	15,000,000	$	15,000,000
	Scour Monitoring Equipment	LS	LS	$	1,000,000	$	1,000,000
	Structural Steel	CWT	99000	$	650	$	64,350,000
	Operator's House	LS	LS	$	750,000	$	750,000
	Mechanical & Electrical	l	LS	$	5,000,000	$	5,000,000
	Fender System	LS	LS	$	500,000	$	1,500,000
	Temporary Construction Access	LS	LS	$	-	$	-
Total Structure Items (2010 Dollars)						*$*	*100,800,000*
Other Items							
	High Towers	EA	3	$	2,000,000	$	6,000,000
Total Other Items (2010 Dollars)						*$*	*6,000,000*
Subtotal (Structure + Other Items)						*$*	*106,800,000*
Miscellaneous Items (Mobilization, Minor Items, etc.) @ 40%						*$*	*42,720,000*
Contingency @ 20%						*$*	*21,360,000*
Signals & Communication						*$*	*4,000,000*
Catenary						*$*	*10,000,000*
Track & Rail						*$*	*9,000,000*
Total Construction Cost						*$*	*193,880,000*
Force Account / RR Protection Services @ 40% of Construction Cost						*$*	*77,552,000*
Engineering Fee @ 15% of Construction Cost						*$*	*29,082,000*
Incidental Expenses @ 20% of Construction Cost						*$*	*38,776,000*
Contingency @ 15% of Construction Cost						*$*	*29,082,000*
					Subtotal (2010 Dollars)	*$*	*368,372,000*
			Assumed Annual Inflation: 6.00%			*$*	*291,326,148*
					Total Project Cost at Midpoint of Construction (2020 Dollars)	***$***	***659,698,148***
					Say:	***$***	***660,000,000*** *

**Cost Adjusted to inflation 6% annually to the midpoint of construction*

Notes:

1. Percentages for Miscellaneous Items, Contingencies, Force Account, Engineering Fee, Incidental Expenses are as directed by CTDOT staff.
2. Costs for Signals & Communication, Catenary, Station Access, and Track & Rail work developed by CTDOT staff.

Alternative IVa – Full Replacement with Trusses

Alternative IVa - Full Replacement with Trusses

Base Year: 2010

Midpoint of Construction: 2020

Preliminary Construction Cost Estimate and Total Project Cost

Item No.	Item	Unit	Quantity	Unit Price	Price
Structure Items					
	Removal of Superstructure	LS	LS	$ 10,500,000	$ 10,500,000
	Cofferdam and Dewatering	LF	1800	$ 1,500	$ 2,700,000
	Removal of Existing Masonry	CY	11000	$ 500	$ 5,500,000
	Drilled Shafts	LF	1700	$ 15,000	$ 25,500,000
	Pier Caps	CY	2078	$ 2,000	$ 4,156,000
	Abutments	LS	LS	$ 2,000,000	$ 2,000,000
	Structural Steel	CWT	90840	$ 650	$ 59,046,000
	Operator's House	LS	LS	$ 750,000	$ 750,000
	Mechanical & Electrical	l	LS	$ 5,000,000	$ 5,000,000
	Fender System	LS	LS	$ 500,000	$ 1,500,000
	Temporary Construction Access	LS	LS	$ 15,000,000	$ 15,000,000
Total Structure Items (2010 Dollars)					*$ 131,652,000*
Other Items					
	High Towers	EA	3	$ 2,000,000	$ 6,000,000
Total Other Items (2010 Dollars)					*$ 6,000,000*
Subtotal (Structure + Other Items)					*$ 137,652,000*
Miscellaneous Items (Mobilization, Minor Items, etc.) @ 40%					*$ 55,060,800*
Contingency @ 20%					*$ 27,530,400*
Signals & Communication					*$ 4,000,000*
Catenary					*$ 12,000,000*
Track & Rail					*$ 9,000,000*
Total Construction Cost					*$ 245,243,200*
Force Account / RR Protection Services @ 40% of Construction Cost					*$ 98,097,280*
Engineering Fee @ 15% of Construction Cost					*$ 36,786,480*
Incidental Expenses @ 20% of Construction Cost					*$ 49,048,640*
Contingency @ 15% of Construction Cost					*$ 36,786,480*
				Subtotal (2010 Dollars)	*$ 465,962,080*
			Assumed Annual Inflation: 6.00%		*$ 368,505,038*
				Total Project Cost at Midpoint of Construction (2020 Dollars)	***$ 834,467,118***
				Say:	***$ 840,000,000 ****

**Cost Adjusted to inflation 6% annually to the midpoint of construction*

Notes:

1. Percentages for Miscellaneous Items, Contingencies, Force Account, Engineering Fee, Incidental Expenses are as directed by CTDOT staff.
2. Costs for Signals & Communication, Catenary, Station Access, and Track & Rail work developed by CTDOT staff.

Alternative IVb – Full Replacement with Deck Girders

Alternative IVb - Full Replacement with Deck Girders

Base Year: 2010

Midpoint of Construction: 2020

Preliminary Construction Cost Estimate and Total Project Cost

Item No.	Item	Unit	Quantity	Unit Price	Price
Structure Items					
	Removal of Superstructure	LS	LS	$ 10,500,000	$ 10,500,000
	Cofferdam and Dewatering	LF	1800	$ 1,500	$ 2,700,000
	Removal of Existing Masonry	CY	11000	$ 500	$ 5,500,000
	Drilled Shafts	LF	2600	$ 15,000	$ 39,000,000
	Pier Caps	CY	2620	$ 2,000	$ 5,240,000
	Abutments	LS	LS	$ 2,000,000	$ 2,000,000
	Structural Steel	CWT	69080	$ 500	$ 34,540,000
	Operator's House	LS	LS	$ 750,000	$ 750,000
	Mechanical & Electrical	LS	LS	$ 5,000,000	$ 5,000,000
	Fender System	LS	LS	$ 500,000	$ 1,500,000
	Temporary Construction Access	LS	LS	$ 15,000,000	$ 15,000,000
Total Structure Items (2010 Dollars)					*$ 121,730,000*
Other Items					
	High Towers	EA	3	$ 2,000,000	$ 6,000,000
Total Other Items (2010 Dollars)					*$ 6,000,000*
Subtotal (Structure + Other Items)					*$ 127,730,000*
Miscellaneous Items (Mobilization, Minor Items, etc.) @ 40%					*$ 51,092,000*
Contingency @ 20%					*$ 25,546,000*
Signals & Communication					*$ 4,000,000*
Catenary					*$ 12,000,000*
Track & Rail					*$ 9,000,000*
Total Construction Cost					*$ 229,368,000*
Force Account / RR Protection Services @ 40% of Construction Cost					*$ 91,747,200*
Engineering Fee @ 15% of Construction Cost					*$ 34,405,200*
Incidental Expenses @ 20% of Construction Cost					*$ 45,873,600*
Contingency @ 15% of Construction Cost					*$ 34,405,200*
				Subtotal (2010 Dollars)	*$ 435,799,200*
			Assumed Annual Inflation: 6.00%		*$ 344,650,793*
				Total Project Cost at Midpoint of Construction (2020 Dollars)	***$ 780,449,993***
				Say:	***$ 790,000,000*** *

**Cost Adjusted to inflation 6% annually to the midpoint of construction*

Notes:

1. Percentages for Miscellaneous Items, Contingencies, Force Account, Engineering Fee, Incidental Expenses are as directed by CTDOT staff.
2. Costs for Signals & Communication, Catenary, Station Access, and Track & Rail work developed by CTDOT staff.

B.) Cost Analysis

A life-cycle cost analysis (LCCA) was developed to provide a financial metric to assist in the evaluation of the alternatives presented in this report. LCCA is recognized by the US Department of Transportation as a means to evaluate two or more alternatives that accomplish the same project objective, enabling the identification of the least cost alternative. LCCA is an appropriate analysis tool offering guidance in selecting among alternatives that offer the same level of service. It differs from a benefit-cost analysis (BCA) in that the BCA evaluates alternatives that have differing project objectives. In the case of the Devon Bridge, the project objective in all alternatives is to ensure continued service of four active MNR and Amtrak tracks over the Housatonic River.

For the analysis of the Devon Bridge, the parameters used in the LCCA are construction, rehabilitation, and maintenance costs, with an operational time period of 75 years. The capital costs developed for the preliminary cost estimates above were used as a basis for the LCCA. Operational costs are not included in the analysis because these costs are assumed to be the same for all alternatives; in all cases one operator is required to control the bridge with personnel from various MNR departments (track, signals, power, etc.) required on site during openings to inspect various components prior to allowing train traffic back on the bridge upon closing. Annual costs for each of the relevant alternatives were developed for each of the above parameters. Then a discount rate was applied to each based on the year the cost would be incurred to yield the total present value of each alternative. The present value was finally converted to an equivalent uniform annual cost (EUAC). It should be noted that the cost analysis is to be used only as a tool to compare the alternatives except Alternative I, which does not have an operational time period and is therefore excluded from the comparison. The cost analysis should not be used for budgetary purposes.

For the purposes of this cost analysis, the following assumptions were made:

- The repairs outlined in Alternative I will have been implemented as the base year condition for each of the alternatives being evaluated;
- The initial rehabilitation cost of Alternative II will be required on a 25-year cycle;
- With Alternative IIIa, Spans 1 through 4 will require future rehabilitation at years 26 and 51, accounted for by applying a percentage of the Alternative II rehabilitation costs at year 26 and 51 (25-year rehabilitation design life);
- Alternatives II, IIIa, & IIIb will require similar on-going substructure maintenance costs due to the retention of the existing substructures;
- Maintenance costs are carried through the year of repair or reconstruction due to construction staging (two tracks of bridge are operating at all times);
- Maintenance costs escalate at a rate of 2% per year in addition to inflation to account for increasing complexity of the required repairs on existing spans to remain. For Alternative II, the maintenance costs are reset to present value upon completion of future year rehabilitations;
- Alternatives IVa and IVb assume a 100-year life span of the bridge, and thus have a residual value at the 75th year equal to 25% of the present value construction cost;
- A real discount (interest) rate of 2.7% was used to develop the base year value of future costs in accordance with OBM Circular No. A-94, Revised 2009, which accounts for the effects of inflation for comparison purposes; and
- User costs (impacts to public) for each of the alternatives were developed based on an assumed daily delay during construction due to construction activities.

A summary of the results of the cost analysis are presented in the following table:

Life Cycle Cost Analysis	
Alternative	Equivalent Uniform Annual Cost (EUAC)
Alternative II	$14,000,000
Alternative IIIa	$14,400,000
Alternative IIIb	$12,100,000
Alternative IVa	$14,900,000
Alternative IVb	$14,000,000

A complete listing of the annualized costs is presented in the following pages for each alternative.

Alternative II – Rehabilitation

Life-Cycle Cost Analysis

EUAC	$ 14,000,000

Discount Rate	2.7%		# Operation Crew		6
Inflation Rate	0.0%		Operation Crew Rate	$	125 per hour
Repair Complexity Rate	2.0%		# Openings per year		100
			Yearly Operation Cost	$	300,000
# Maintenance Crew	4				
Maintenance Crew Rate	$ 125 per hour		Construction Duration		12 months
Maintenance Required Every	2 weeks		Average Delay due to Construction		5 minutes/day
Yearly Maintenance Labor Cost	$ 104,000		Value of Time	$	40 per hour
Materials and Equipment cost @ 25%	$ 26,000		Ridership		6300 per weekday (one way)
Total Yearly Maintenance Cost	$ 130,000				

Year	Construction/Repair		Maintenance		Operations		User Costs	
	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost
1	$ 165,000,000	$ 165,000,000	$ 130,000	$ 130,000	$ 300,000	$ 300,000	$ 5,460,000	$ 5,460,000
2		$ -	$ 130,000	$ 126,582	$ 300,000	$ 292,113		
3		$ -	$ 130,000	$ 123,254	$ 300,000	$ 284,433		
4		$ -	$ 130,000	$ 120,014	$ 300,000	$ 276,955		
5		$ -	$ 130,000	$ 116,859	$ 300,000	$ 269,674		
6		$ -	$ 130,000	$ 113,787	$ 300,000	$ 262,584		
7		$ -	$ 130,000	$ 110,795	$ 300,000	$ 255,681		
8		$ -	$ 130,000	$ 107,882	$ 300,000	$ 248,959		
9		$ -	$ 130,000	$ 105,046	$ 300,000	$ 242,414		
10		$ -	$ 130,000	$ 102,284	$ 300,000	$ 236,041		
11		$ -	$ 130,000	$ 99,595	$ 300,000	$ 229,835		
12		$ -	$ 130,000	$ 96,977	$ 300,000	$ 223,793		
13		$ -	$ 130,000	$ 94,427	$ 300,000	$ 217,909		
14		$ -	$ 130,000	$ 91,945	$ 300,000	$ 212,181		
15		$ -	$ 130,000	$ 89,528	$ 300,000	$ 206,602		
16		$ -	$ 130,000	$ 87,174	$ 300,000	$ 201,171		
17		$ -	$ 130,000	$ 84,882	$ 300,000	$ 195,882		
18		$ -	$ 130,000	$ 82,651	$ 300,000	$ 190,732		
19		$ -	$ 130,000	$ 80,478	$ 300,000	$ 185,718		
20		$ -	$ 130,000	$ 78,362	$ 300,000	$ 180,835		
21		$ -	$ 130,000	$ 76,302	$ 300,000	$ 176,081		
22		$ -	$ 130,000	$ 74,296	$ 300,000	$ 171,452		
23		$ -	$ 130,000	$ 72,343	$ 300,000	$ 166,944		
24		$ -	$ 130,000	$ 70,441	$ 300,000	$ 162,555		
25		$ -	$ 130,000	$ 68,589	$ 300,000	$ 158,282		
26	$ 270,699,989	$ 139,067,981	$ 130,000	$ 66,786	$ 300,000	$ 154,120	$ 5,460,000	$ 2,804,992
27		$ -	$ 130,000	$ 65,030	$ 300,000	$ 150,069		
28		$ -	$ 130,000	$ 63,320	$ 300,000	$ 146,123		
29		$ -	$ 130,000	$ 61,655	$ 300,000	$ 142,282		
30		$ -	$ 130,000	$ 60,034	$ 300,000	$ 138,541		
31		$ -	$ 130,000	$ 58,456	$ 300,000	$ 134,899		
32		$ -	$ 130,000	$ 56,919	$ 300,000	$ 131,352		
33		$ -	$ 130,000	$ 55,423	$ 300,000	$ 127,899		
34		$ -	$ 130,000	$ 53,966	$ 300,000	$ 124,536		
35		$ -	$ 130,000	$ 52,547	$ 300,000	$ 121,262		
36		$ -	$ 130,000	$ 51,166	$ 300,000	$ 118,074		
37		$ -	$ 130,000	$ 49,820	$ 300,000	$ 114,970		
38		$ -	$ 130,000	$ 48,511	$ 300,000	$ 111,948		
39		$ -	$ 130,000	$ 47,235	$ 300,000	$ 109,004		
40		$ -	$ 130,000	$ 45,993	$ 300,000	$ 106,139		
41		$ -	$ 130,000	$ 44,784	$ 300,000	$ 103,348		
42		$ -	$ 130,000	$ 43,607	$ 300,000	$ 100,631		
43		$ -	$ 130,000	$ 42,460	$ 300,000	$ 97,986		
44		$ -	$ 130,000	$ 41,344	$ 300,000	$ 95,410		
45		$ -	$ 130,000	$ 40,257	$ 300,000	$ 92,901		
46		$ -	$ 130,000	$ 39,199	$ 300,000	$ 90,459		
47		$ -	$ 130,000	$ 38,168	$ 300,000	$ 88,081		
48		$ -	$ 130,000	$ 37,165	$ 300,000	$ 85,765		
49		$ -	$ 130,000	$ 36,188	$ 300,000	$ 83,510		
50		$ -	$ 130,000	$ 35,236	$ 300,000	$ 81,315		
51	$ 444,112,025	$ 117,211,535	$ 130,000	$ 34,310	$ 300,000	$ 79,177	$ 5,460,000	$ 1,441,022
52		$ -	$ 130,000	$ 33,408	$ 300,000	$ 77,095		
53		$ -	$ 130,000	$ 32,530	$ 300,000	$ 75,069		
54		$ -	$ 130,000	$ 31,675	$ 300,000	$ 73,095		
55		$ -	$ 130,000	$ 30,842	$ 300,000	$ 71,173		
56		$ -	$ 130,000	$ 30,031	$ 300,000	$ 69,302		
57		$ -	$ 130,000	$ 29,241	$ 300,000	$ 67,480		
58		$ -	$ 130,000	$ 28,473	$ 300,000	$ 65,706		
59		$ -	$ 130,000	$ 27,724	$ 300,000	$ 63,979		
60		$ -	$ 130,000	$ 26,995	$ 300,000	$ 62,297		
61		$ -	$ 130,000	$ 26,286	$ 300,000	$ 60,659		
62		$ -	$ 130,000	$ 25,594	$ 300,000	$ 59,064		
63		$ -	$ 130,000	$ 24,922	$ 300,000	$ 57,511		
64		$ -	$ 130,000	$ 24,266	$ 300,000	$ 55,999		
65		$ -	$ 130,000	$ 23,628	$ 300,000	$ 54,527		
66		$ -	$ 130,000	$ 23,007	$ 300,000	$ 53,094		
67		$ -	$ 130,000	$ 22,402	$ 300,000	$ 51,698		
68		$ -	$ 130,000	$ 21,813	$ 300,000	$ 50,339		
69		$ -	$ 130,000	$ 21,240	$ 300,000	$ 49,015		
70		$ -	$ 130,000	$ 20,682	$ 300,000	$ 47,727		
71		$ -	$ 130,000	$ 20,138	$ 300,000	$ 46,472		
72		$ -	$ 130,000	$ 19,608	$ 300,000	$ 45,250		
73		$ -	$ 130,000	$ 19,093	$ 300,000	$ 44,060		
74		$ -	$ 130,000	$ 18,591	$ 300,000	$ 42,902		
75		$ -	$ 130,000	$ 18,102	$ 300,000	$ 41,774		$ -
Total		$ 421,279,515		$ 4,274,364		$ 9,863,917		$ 9,706,013
NPV		$ 445,123,810						
EUAC		$ 13,903,466						

Alternative IIIa – Partial Superstructure Replacement

Life-Cycle Cost Analysis

EUAC	$ 14,400,000

Discount Rate		2.7%	# Operation Crew		6
Inflation Rate		0.0%	Operation Crew Rate	$	125 per hour
Repair Complexity Rate		2.0%	# Openings per year		100
			Yearly Operation Cost	$	300,000
# Maintenance Crew		4			
Maintenance Crew Rate	$	125 per hour	Construction Duration		24 months
Maintenance Required Every		5 weeks	Average Delay due to Construction		5 minutes/day
Yearly Maintenance Labor Cost	$	41,600	Value of Time	$	40 per hour
Materials and Equipment cost @ 25%	$	10,400	Ridership		6300 per weekday (one way)
Total Yearly Maintenance Cost	$	52,000			

Year	Construction/Repair		Maintenance		Operations		User Costs	
	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost
1	$ 325,000,000	$ 325,000,000	$ 52,000	$ 52,000	$ 300,000	$ 300,000	$ 5,460,000	$ 5,460,000
2		$ -	$ 52,000	$ 50,633	$ 300,000	$ 292,113	$ 5,460,000	$ 5,316,456
3		$ -	$ 52,000	$ 49,302	$ 300,000	$ 284,433		
4		$ -	$ 52,000	$ 48,006	$ 300,000	$ 276,955		
5		$ -	$ 52,000	$ 46,744	$ 300,000	$ 269,674		
6		$ -	$ 52,000	$ 45,515	$ 300,000	$ 262,584		
7		$ -	$ 52,000	$ 44,318	$ 300,000	$ 255,681		
8		$ -	$ 52,000	$ 43,153	$ 300,000	$ 248,959		
9		$ -	$ 52,000	$ 42,018	$ 300,000	$ 242,414		
10		$ -	$ 52,000	$ 40,914	$ 300,000	$ 236,041		
11		$ -	$ 52,000	$ 39,838	$ 300,000	$ 229,835		
12		$ -	$ 52,000	$ 38,791	$ 300,000	$ 223,793		
13		$ -	$ 52,000	$ 37,771	$ 300,000	$ 217,909		
14		$ -	$ 52,000	$ 36,778	$ 300,000	$ 212,181		
15		$ -	$ 52,000	$ 35,811	$ 300,000	$ 206,602		
16		$ -	$ 52,000	$ 34,870	$ 300,000	$ 201,171		
17		$ -	$ 52,000	$ 33,953	$ 300,000	$ 195,882		
18		$ -	$ 52,000	$ 33,060	$ 300,000	$ 190,732		
19		$ -	$ 52,000	$ 32,191	$ 300,000	$ 185,718		
20		$ -	$ 52,000	$ 31,345	$ 300,000	$ 180,835		
21		$ -	$ 52,000	$ 30,521	$ 300,000	$ 176,081		
22		$ -	$ 52,000	$ 29,718	$ 300,000	$ 171,452		
23		$ -	$ 52,000	$ 28,937	$ 300,000	$ 166,944		
24		$ -	$ 52,000	$ 28,176	$ 300,000	$ 162,555		
25		$ -	$ 52,000	$ 27,435	$ 300,000	$ 158,282		
26	$ 108,279,996	$ 55,627,192	$ 52,000	$ 26,714	$ 300,000	$ 154,120	$ 5,460,000	$ 2,804,992
27		$ -	$ 52,000	$ 26,012	$ 300,000	$ 150,069	$ 5,460,000	$ 2,731,248
28		$ -	$ 52,000	$ 25,328	$ 300,000	$ 146,123		
29		$ -	$ 52,000	$ 24,662	$ 300,000	$ 142,282		
30		$ -	$ 52,000	$ 24,014	$ 300,000	$ 138,541		
31		$ -	$ 52,000	$ 23,382	$ 300,000	$ 134,899		
32		$ -	$ 52,000	$ 22,768	$ 300,000	$ 131,352		
33		$ -	$ 52,000	$ 22,169	$ 300,000	$ 127,899		
34		$ -	$ 52,000	$ 21,586	$ 300,000	$ 124,536		
35		$ -	$ 52,000	$ 21,019	$ 300,000	$ 121,262		
36		$ -	$ 52,000	$ 20,466	$ 300,000	$ 118,074		
37		$ -	$ 52,000	$ 19,928	$ 300,000	$ 114,970		
38		$ -	$ 52,000	$ 19,404	$ 300,000	$ 111,948		
39		$ -	$ 52,000	$ 18,894	$ 300,000	$ 109,004		
40		$ -	$ 52,000	$ 18,397	$ 300,000	$ 106,139		
41		$ -	$ 52,000	$ 17,914	$ 300,000	$ 103,348		
42		$ -	$ 52,000	$ 17,443	$ 300,000	$ 100,631		
43		$ -	$ 52,000	$ 16,984	$ 300,000	$ 97,986		
44		$ -	$ 52,000	$ 16,538	$ 300,000	$ 95,410		
45		$ -	$ 52,000	$ 16,103	$ 300,000	$ 92,901		
46		$ -	$ 52,000	$ 15,680	$ 300,000	$ 90,459		
47		$ -	$ 52,000	$ 15,267	$ 300,000	$ 88,081		
48		$ -	$ 52,000	$ 14,866	$ 300,000	$ 85,765		
49		$ -	$ 52,000	$ 14,475	$ 300,000	$ 83,510		
50		$ -	$ 52,000	$ 14,095	$ 300,000	$ 81,315		
51	$ 177,644,810	$ 46,884,614	$ 52,000	$ 13,724	$ 300,000	$ 79,177	$ 5,460,000	$ 1,441,022
52		$ -	$ 52,000	$ 13,363	$ 300,000	$ 77,095	$ 5,460,000	$ 1,403,137
53		$ -	$ 52,000	$ 13,012	$ 300,000	$ 75,069		
54		$ -	$ 52,000	$ 12,670	$ 300,000	$ 73,095		
55		$ -	$ 52,000	$ 12,337	$ 300,000	$ 71,173		
56		$ -	$ 52,000	$ 12,012	$ 300,000	$ 69,302		
57		$ -	$ 52,000	$ 11,697	$ 300,000	$ 67,480		
58		$ -	$ 52,000	$ 11,389	$ 300,000	$ 65,706		
59		$ -	$ 52,000	$ 11,090	$ 300,000	$ 63,979		
60		$ -	$ 52,000	$ 10,798	$ 300,000	$ 62,297		
61		$ -	$ 52,000	$ 10,514	$ 300,000	$ 60,659		
62		$ -	$ 52,000	$ 10,238	$ 300,000	$ 59,064		
63		$ -	$ 52,000	$ 9,969	$ 300,000	$ 57,511		
64		$ -	$ 52,000	$ 9,707	$ 300,000	$ 55,999		
65		$ -	$ 52,000	$ 9,451	$ 300,000	$ 54,527		
66		$ -	$ 52,000	$ 9,203	$ 300,000	$ 53,094		
67		$ -	$ 52,000	$ 8,961	$ 300,000	$ 51,698		
68		$ -	$ 52,000	$ 8,725	$ 300,000	$ 50,339		
69		$ -	$ 52,000	$ 8,496	$ 300,000	$ 49,015		
70		$ -	$ 52,000	$ 8,273	$ 300,000	$ 47,727		
71		$ -	$ 52,000	$ 8,055	$ 300,000	$ 46,472		
72		$ -	$ 52,000	$ 7,843	$ 300,000	$ 45,250		
73		$ -	$ 52,000	$ 7,637	$ 300,000	$ 44,060		
74		$ -	$ 52,000	$ 7,436	$ 300,000	$ 42,902		
75		$ -	$ 52,000	$ 7,241	$ 300,000	$ 41,774		$ -
Total		$ 427,511,806		$ 1,709,746		$ 9,863,917		$ 19,156,853
NPV		$ 458,242,323						
EUAC		$ 14,313,224						

Alternative IIIb – Full Superstructure Replacement

Life-Cycle Cost Analysis

EUAC $ 12,100,000

Discount Rate		2.7%
Inflation Rate		0.0%
Repair Complexity Rate		0.0%
# Maintenance Crew		4
Maintenance Crew Rate	$	125 per hour
Maintenance Required Every		18 weeks
Yearly Maintenance Labor Cost	$	11,556
Materials and Equipment cost @ 25%	$	2,889
Total Yearly Maintenance Cost	$	14,444

# Operation Crew		2
Operation Crew Rate	$	125 per hour
# Openings per year		100
Yearly Operation Cost	$	100,000
Construction Duration		24 months
Average Delay due to Construction		5 minutes/day
Value of Time	$	40 per hour
Ridership		6300 per weekday (one way)

Year	Construction/Repair		Maintenance		Operations		User Costs	
	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost
1	$ 370,000,000	$ 370,000,000	$ 14,444	$ 14,444	$ 100,000	$ 100,000	$ 5,460,000	$ 5,460,000
2		$ -	$ 14,444	$ 14,065	$ 100,000	$ 97,371	$ 5,460,000	$ 5,316,456
3		$ -	$ 14,444	$ 13,695	$ 100,000	$ 94,811		
4		$ -	$ 14,444	$ 13,335	$ 100,000	$ 92,318		
5		$ -	$ 14,444	$ 12,984	$ 100,000	$ 89,891		
6		$ -	$ 14,444	$ 12,643	$ 100,000	$ 87,528		
7		$ -	$ 14,444	$ 12,311	$ 100,000	$ 85,227		
8		$ -	$ 14,444	$ 11,987	$ 100,000	$ 82,986		
9		$ -	$ 14,444	$ 11,672	$ 100,000	$ 80,805		
10		$ -	$ 14,444	$ 11,365	$ 100,000	$ 78,680		
11		$ -	$ 14,444	$ 11,066	$ 100,000	$ 76,612		
12		$ -	$ 14,444	$ 10,775	$ 100,000	$ 74,598		
13		$ -	$ 14,444	$ 10,492	$ 100,000	$ 72,636		
14		$ -	$ 14,444	$ 10,216	$ 100,000	$ 70,727		
15		$ -	$ 14,444	$ 9,948	$ 100,000	$ 68,867		
16		$ -	$ 14,444	$ 9,686	$ 100,000	$ 67,057		
17		$ -	$ 14,444	$ 9,431	$ 100,000	$ 65,294		
18		$ -	$ 14,444	$ 9,183	$ 100,000	$ 63,577		
19		$ -	$ 14,444	$ 8,942	$ 100,000	$ 61,906		
20		$ -	$ 14,444	$ 8,707	$ 100,000	$ 60,278		
21		$ -	$ 14,444	$ 8,478	$ 100,000	$ 58,694		
22		$ -	$ 14,444	$ 8,255	$ 100,000	$ 57,151		
23		$ -	$ 14,444	$ 8,038	$ 100,000	$ 55,648		
24		$ -	$ 14,444	$ 7,827	$ 100,000	$ 54,185		
25		$ -	$ 14,444	$ 7,621	$ 100,000	$ 52,761		
26		$ -	$ 14,444	$ 7,421	$ 100,000	$ 51,373		
27		$ -	$ 14,444	$ 7,226	$ 100,000	$ 50,023		
28		$ -	$ 14,444	$ 7,036	$ 100,000	$ 48,708		
29		$ -	$ 14,444	$ 6,851	$ 100,000	$ 47,427		
30		$ -	$ 14,444	$ 6,670	$ 100,000	$ 46,180		
31		$ -	$ 14,444	$ 6,495	$ 100,000	$ 44,966		
32		$ -	$ 14,444	$ 6,324	$ 100,000	$ 43,784		
33		$ -	$ 14,444	$ 6,158	$ 100,000	$ 42,633		
34		$ -	$ 14,444	$ 5,996	$ 100,000	$ 41,512		
35		$ -	$ 14,444	$ 5,839	$ 100,000	$ 40,421		
36		$ -	$ 14,444	$ 5,685	$ 100,000	$ 39,358		
37		$ -	$ 14,444	$ 5,536	$ 100,000	$ 38,323		
38		$ -	$ 14,444	$ 5,390	$ 100,000	$ 37,316		
39		$ -	$ 14,444	$ 5,248	$ 100,000	$ 36,335		
40		$ -	$ 14,444	$ 5,110	$ 100,000	$ 35,380		
41		$ -	$ 14,444	$ 4,976	$ 100,000	$ 34,449		
42		$ -	$ 14,444	$ 4,845	$ 100,000	$ 33,544		
43		$ -	$ 14,444	$ 4,718	$ 100,000	$ 32,662		
44		$ -	$ 14,444	$ 4,594	$ 100,000	$ 31,803		
45		$ -	$ 14,444	$ 4,473	$ 100,000	$ 30,967		
46		$ -	$ 14,444	$ 4,355	$ 100,000	$ 30,153		
47		$ -	$ 14,444	$ 4,241	$ 100,000	$ 29,360		
48		$ -	$ 14,444	$ 4,129	$ 100,000	$ 28,588		
49		$ -	$ 14,444	$ 4,021	$ 100,000	$ 27,837		
50		$ -	$ 14,444	$ 3,915	$ 100,000	$ 27,105		
51		$ -	$ 14,444	$ 3,812	$ 100,000	$ 26,392		
52		$ -	$ 14,444	$ 3,712	$ 100,000	$ 25,698		
53		$ -	$ 14,444	$ 3,614	$ 100,000	$ 25,023		
54		$ -	$ 14,444	$ 3,519	$ 100,000	$ 24,365		
55		$ -	$ 14,444	$ 3,427	$ 100,000	$ 23,724		
56		$ -	$ 14,444	$ 3,337	$ 100,000	$ 23,101		
57		$ -	$ 14,444	$ 3,249	$ 100,000	$ 22,493		
58		$ -	$ 14,444	$ 3,164	$ 100,000	$ 21,902		
59		$ -	$ 14,444	$ 3,080	$ 100,000	$ 21,326		
60		$ -	$ 14,444	$ 2,999	$ 100,000	$ 20,766		
61		$ -	$ 14,444	$ 2,921	$ 100,000	$ 20,220		
62		$ -	$ 14,444	$ 2,844	$ 100,000	$ 19,688		
63		$ -	$ 14,444	$ 2,769	$ 100,000	$ 19,170		
64		$ -	$ 14,444	$ 2,696	$ 100,000	$ 18,666		
65		$ -	$ 14,444	$ 2,625	$ 100,000	$ 18,176		
66		$ -	$ 14,444	$ 2,556	$ 100,000	$ 17,698		
67		$ -	$ 14,444	$ 2,489	$ 100,000	$ 17,233		
68		$ -	$ 14,444	$ 2,424	$ 100,000	$ 16,780		
69		$ -	$ 14,444	$ 2,360	$ 100,000	$ 16,338		
70		$ -	$ 14,444	$ 2,298	$ 100,000	$ 15,909		
71		$ -	$ 14,444	$ 2,238	$ 100,000	$ 15,491		
72		$ -	$ 14,444	$ 2,179	$ 100,000	$ 15,083		
73		$ -	$ 14,444	$ 2,121	$ 100,000	$ 14,687		
74		$ -	$ 14,444	$ 2,066	$ 100,000	$ 14,301		
75		$ -	$ 14,444	$ 2,011	$ 100,000	$ 13,925		$ -
Total		$ 370,000,000		$ 474,929		$ 3,287,972		$ 10,776,456
NPV		$ 384,539,358						
EUAC		$ 12,011,108						

Alternative IVa – Full Replacement with Trusses

Life-Cycle Cost Analysis

EUAC	$ 14,900,000

Discount Rate	2.7%	# Operation Crew	1
Inflation Rate	0.0%	Operation Crew Rate	$ 125 per hour
Repair Complexity Rate	0.0%	# Openings per year	100
		Yearly Operation Cost	$ 50,000
# Maintenance Crew	4		
Maintenance Crew Rate	$ 125 per hour	Construction Duration	48 months
Maintenance Required Every	26 weeks	Average Delay due to Construction	5 minutes/day
Yearly Maintenance Labor Cost	$ 8,000	Value of Time	$ 40 per hour
Materials and Equipment cost @ 25%	$ 2,000	Ridership	6300 per weekday (one way)
Total Yearly Maintenance Cost	$ 10,000		

Year	Construction/Repair		Maintenance		Operations		User Costs	
	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost
1	$ 470,000,000	$ 470,000,000	$ 10,000	$ 10,000	$ 50,000	$ 50,000	$ 5,460,000	$ 5,460,000
2		$ -	$ 10,000	$ 9,737	$ 50,000	$ 48,685	$ 5,460,000	$ 5,316,456
3		$ -	$ 10,000	$ 9,481	$ 50,000	$ 47,406	$ 5,460,000	$ 5,176,685
4		$ -	$ 10,000	$ 9,232	$ 50,000	$ 46,159	$ 5,460,000	$ 5,040,589
5		$ -	$ 10,000	$ 8,989	$ 50,000	$ 44,946		
6		$ -	$ 10,000	$ 8,753	$ 50,000	$ 43,764		
7		$ -	$ 10,000	$ 8,523	$ 50,000	$ 42,614		
8		$ -	$ 10,000	$ 8,299	$ 50,000	$ 41,493		
9		$ -	$ 10,000	$ 8,080	$ 50,000	$ 40,402		
10		$ -	$ 10,000	$ 7,868	$ 50,000	$ 39,340		
11		$ -	$ 10,000	$ 7,661	$ 50,000	$ 38,306		
12		$ -	$ 10,000	$ 7,460	$ 50,000	$ 37,299		
13		$ -	$ 10,000	$ 7,264	$ 50,000	$ 36,318		
14		$ -	$ 10,000	$ 7,073	$ 50,000	$ 35,363		
15		$ -	$ 10,000	$ 6,887	$ 50,000	$ 34,434		
16		$ -	$ 10,000	$ 6,706	$ 50,000	$ 33,528		
17		$ -	$ 10,000	$ 6,529	$ 50,000	$ 32,647		
18		$ -	$ 10,000	$ 6,358	$ 50,000	$ 31,789		
19		$ -	$ 10,000	$ 6,191	$ 50,000	$ 30,953		
20		$ -	$ 10,000	$ 6,028	$ 50,000	$ 30,139		
21		$ -	$ 10,000	$ 5,869	$ 50,000	$ 29,347		
22		$ -	$ 10,000	$ 5,715	$ 50,000	$ 28,575		
23		$ -	$ 10,000	$ 5,565	$ 50,000	$ 27,824		
24		$ -	$ 10,000	$ 5,419	$ 50,000	$ 27,093		
25		$ -	$ 10,000	$ 5,276	$ 50,000	$ 26,380		
26		$ -	$ 10,000	$ 5,137	$ 50,000	$ 25,687		
27		$ -	$ 10,000	$ 5,002	$ 50,000	$ 25,011		
28		$ -	$ 10,000	$ 4,871	$ 50,000	$ 24,354		
29		$ -	$ 10,000	$ 4,743	$ 50,000	$ 23,714		
30		$ -	$ 10,000	$ 4,618	$ 50,000	$ 23,090		
31		$ -	$ 10,000	$ 4,497	$ 50,000	$ 22,483		
32		$ -	$ 10,000	$ 4,378	$ 50,000	$ 21,892		
33		$ -	$ 10,000	$ 4,263	$ 50,000	$ 21,316		
34		$ -	$ 10,000	$ 4,151	$ 50,000	$ 20,756		
35		$ -	$ 10,000	$ 4,042	$ 50,000	$ 20,210		
36		$ -	$ 10,000	$ 3,936	$ 50,000	$ 19,679		
37		$ -	$ 10,000	$ 3,832	$ 50,000	$ 19,162		
38		$ -	$ 10,000	$ 3,732	$ 50,000	$ 18,658		
39		$ -	$ 10,000	$ 3,633	$ 50,000	$ 18,167		
40		$ -	$ 10,000	$ 3,538	$ 50,000	$ 17,690		
41		$ -	$ 10,000	$ 3,445	$ 50,000	$ 17,225		
42		$ -	$ 10,000	$ 3,354	$ 50,000	$ 16,772		
43		$ -	$ 10,000	$ 3,266	$ 50,000	$ 16,331		
44		$ -	$ 10,000	$ 3,180	$ 50,000	$ 15,902		
45		$ -	$ 10,000	$ 3,097	$ 50,000	$ 15,484		
46		$ -	$ 10,000	$ 3,015	$ 50,000	$ 15,076		
47		$ -	$ 10,000	$ 2,936	$ 50,000	$ 14,680		
48		$ -	$ 10,000	$ 2,859	$ 50,000	$ 14,294		
49		$ -	$ 10,000	$ 2,784	$ 50,000	$ 13,918		
50		$ -	$ 10,000	$ 2,710	$ 50,000	$ 13,552		
51		$ -	$ 10,000	$ 2,639	$ 50,000	$ 13,196		
52		$ -	$ 10,000	$ 2,570	$ 50,000	$ 12,849		
53		$ -	$ 10,000	$ 2,502	$ 50,000	$ 12,511		
54		$ -	$ 10,000	$ 2,437	$ 50,000	$ 12,183		
55		$ -	$ 10,000	$ 2,372	$ 50,000	$ 11,862		
56		$ -	$ 10,000	$ 2,310	$ 50,000	$ 11,550		
57		$ -	$ 10,000	$ 2,249	$ 50,000	$ 11,247		
58		$ -	$ 10,000	$ 2,190	$ 50,000	$ 10,951		
59		$ -	$ 10,000	$ 2,133	$ 50,000	$ 10,663		
60		$ -	$ 10,000	$ 2,077	$ 50,000	$ 10,383		
61		$ -	$ 10,000	$ 2,022	$ 50,000	$ 10,110		
62		$ -	$ 10,000	$ 1,969	$ 50,000	$ 9,844		
63		$ -	$ 10,000	$ 1,917	$ 50,000	$ 9,585		
64		$ -	$ 10,000	$ 1,867	$ 50,000	$ 9,333		
65		$ -	$ 10,000	$ 1,818	$ 50,000	$ 9,088		
66		$ -	$ 10,000	$ 1,770	$ 50,000	$ 8,849		
67		$ -	$ 10,000	$ 1,723	$ 50,000	$ 8,616		
68		$ -	$ 10,000	$ 1,678	$ 50,000	$ 8,390		
69		$ -	$ 10,000	$ 1,634	$ 50,000	$ 8,169		
70		$ -	$ 10,000	$ 1,591	$ 50,000	$ 7,954		
71		$ -	$ 10,000	$ 1,549	$ 50,000	$ 7,745		
72		$ -	$ 10,000	$ 1,508	$ 50,000	$ 7,542		
73		$ -	$ 10,000	$ 1,469	$ 50,000	$ 7,343		
74		$ -	$ 10,000	$ 1,430	$ 50,000	$ 7,150		
75	$ (117,500,000)	$ (16,361,572)	$ 10,000	$ 1,392	$ 50,000	$ 6,962		$ -
Total		$ 453,638,428		$ 328,797		$ 1,643,986		$ 20,993,730
NPV		$ 476,604,941						
EUAC		$ 14,886,781						

Alternative IVb – Full Replacement with Deck Girders

Life-Cycle Cost Analysis

EUAC	$ 14,000,000

Discount Rate	2.7%	# Operation Crew	1
Inflation Rate	0.0%	Operation Crew Rate	$ 125 per hour
Repair Complexity Rate	0.0%	# Openings per year	100
		Yearly Operation Cost	$ 50,000
# Maintenance Crew	4		
Maintenance Crew Rate	$ 125 per hour	Construction Duration	48 months
Maintenance Required Every	26 weeks	Average Delay due to Construction	5 minutes/day
Yearly Maintenance Labor Cost	$ 8,000	Value of Time	$ 40 per hour
Materials and Equipment cost @ 25%	$ 2,000	Ridership	6300 per weekday (one way)
Total Yearly Maintenance Cost	$ 10,000		

Year	Construction/Repair		Maintenance		Operations		User Costs	
	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost	Base Cost	Discounted Cost
1	$ 440,000,000	$ 440,000,000	$ 10,000	$ 10,000	$ 50,000	$ 50,000	$ 5,460,000	$ 5,460,000
2		$ -	$ 10,000	$ 9,737	$ 50,000	$ 48,685	$ 5,460,000	$ 5,316,456
3		$ -	$ 10,000	$ 9,481	$ 50,000	$ 47,406	$ 5,460,000	$ 5,176,685
4		$ -	$ 10,000	$ 9,232	$ 50,000	$ 46,159	$ 5,460,000	$ 5,040,589
5		$ -	$ 10,000	$ 8,989	$ 50,000	$ 44,946		
6		$ -	$ 10,000	$ 8,753	$ 50,000	$ 43,764		
7		$ -	$ 10,000	$ 8,523	$ 50,000	$ 42,614		
8		$ -	$ 10,000	$ 8,299	$ 50,000	$ 41,493		
9		$ -	$ 10,000	$ 8,080	$ 50,000	$ 40,402		
10		$ -	$ 10,000	$ 7,868	$ 50,000	$ 39,340		
11		$ -	$ 10,000	$ 7,661	$ 50,000	$ 38,306		
12		$ -	$ 10,000	$ 7,460	$ 50,000	$ 37,299		
13		$ -	$ 10,000	$ 7,264	$ 50,000	$ 36,318		
14		$ -	$ 10,000	$ 7,073	$ 50,000	$ 35,363		
15		$ -	$ 10,000	$ 6,887	$ 50,000	$ 34,434		
16		$ -	$ 10,000	$ 6,706	$ 50,000	$ 33,528		
17		$ -	$ 10,000	$ 6,529	$ 50,000	$ 32,647		
18		$ -	$ 10,000	$ 6,358	$ 50,000	$ 31,789		
19		$ -	$ 10,000	$ 6,191	$ 50,000	$ 30,953		
20		$ -	$ 10,000	$ 6,028	$ 50,000	$ 30,139		
21		$ -	$ 10,000	$ 5,869	$ 50,000	$ 29,347		
22		$ -	$ 10,000	$ 5,715	$ 50,000	$ 28,575		
23		$ -	$ 10,000	$ 5,565	$ 50,000	$ 27,824		
24		$ -	$ 10,000	$ 5,419	$ 50,000	$ 27,093		
25		$ -	$ 10,000	$ 5,276	$ 50,000	$ 26,380		
26		$ -	$ 10,000	$ 5,137	$ 50,000	$ 25,687		
27		$ -	$ 10,000	$ 5,002	$ 50,000	$ 25,011		
28		$ -	$ 10,000	$ 4,871	$ 50,000	$ 24,354		
29		$ -	$ 10,000	$ 4,743	$ 50,000	$ 23,714		
30		$ -	$ 10,000	$ 4,618	$ 50,000	$ 23,090		
31		$ -	$ 10,000	$ 4,497	$ 50,000	$ 22,483		
32		$ -	$ 10,000	$ 4,378	$ 50,000	$ 21,892		
33		$ -	$ 10,000	$ 4,263	$ 50,000	$ 21,316		
34		$ -	$ 10,000	$ 4,151	$ 50,000	$ 20,756		
35		$ -	$ 10,000	$ 4,042	$ 50,000	$ 20,210		
36		$ -	$ 10,000	$ 3,936	$ 50,000	$ 19,679		
37		$ -	$ 10,000	$ 3,832	$ 50,000	$ 19,162		
38		$ -	$ 10,000	$ 3,732	$ 50,000	$ 18,658		
39		$ -	$ 10,000	$ 3,633	$ 50,000	$ 18,167		
40		$ -	$ 10,000	$ 3,538	$ 50,000	$ 17,690		
41		$ -	$ 10,000	$ 3,445	$ 50,000	$ 17,225		
42		$ -	$ 10,000	$ 3,354	$ 50,000	$ 16,772		
43		$ -	$ 10,000	$ 3,266	$ 50,000	$ 16,331		
44		$ -	$ 10,000	$ 3,180	$ 50,000	$ 15,902		
45		$ -	$ 10,000	$ 3,097	$ 50,000	$ 15,484		
46		$ -	$ 10,000	$ 3,015	$ 50,000	$ 15,076		
47		$ -	$ 10,000	$ 2,936	$ 50,000	$ 14,680		
48		$ -	$ 10,000	$ 2,859	$ 50,000	$ 14,294		
49		$ -	$ 10,000	$ 2,784	$ 50,000	$ 13,918		
50		$ -	$ 10,000	$ 2,710	$ 50,000	$ 13,552		
51		$ -	$ 10,000	$ 2,639	$ 50,000	$ 13,196		
52		$ -	$ 10,000	$ 2,570	$ 50,000	$ 12,849		
53		$ -	$ 10,000	$ 2,502	$ 50,000	$ 12,511		
54		$ -	$ 10,000	$ 2,437	$ 50,000	$ 12,183		
55		$ -	$ 10,000	$ 2,372	$ 50,000	$ 11,862		
56		$ -	$ 10,000	$ 2,310	$ 50,000	$ 11,550		
57		$ -	$ 10,000	$ 2,249	$ 50,000	$ 11,247		
58		$ -	$ 10,000	$ 2,190	$ 50,000	$ 10,951		
59		$ -	$ 10,000	$ 2,133	$ 50,000	$ 10,663		
60		$ -	$ 10,000	$ 2,077	$ 50,000	$ 10,383		
61		$ -	$ 10,000	$ 2,022	$ 50,000	$ 10,110		
62		$ -	$ 10,000	$ 1,969	$ 50,000	$ 9,844		
63		$ -	$ 10,000	$ 1,917	$ 50,000	$ 9,585		
64		$ -	$ 10,000	$ 1,867	$ 50,000	$ 9,333		
65		$ -	$ 10,000	$ 1,818	$ 50,000	$ 9,088		
66		$ -	$ 10,000	$ 1,770	$ 50,000	$ 8,849		
67		$ -	$ 10,000	$ 1,723	$ 50,000	$ 8,616		
68		$ -	$ 10,000	$ 1,678	$ 50,000	$ 8,390		
69		$ -	$ 10,000	$ 1,634	$ 50,000	$ 8,169		
70		$ -	$ 10,000	$ 1,591	$ 50,000	$ 7,954		
71		$ -	$ 10,000	$ 1,549	$ 50,000	$ 7,745		
72		$ -	$ 10,000	$ 1,508	$ 50,000	$ 7,542		
73		$ -	$ 10,000	$ 1,469	$ 50,000	$ 7,343		
74		$ -	$ 10,000	$ 1,430	$ 50,000	$ 7,150		
75	$ (110,000,000)	$ (15,317,217)	$ 10,000	$ 1,392	$ 50,000	$ 6,962		$ -
Total		$ 424,682,783		$ 328,797		$ 1,643,986		$ 20,993,730
NPV		$ 447,649,297						
EUAC		$ 13,982,350						

XIII. Recommendations

A review of the results of the in-depth inspection report and as-built and as-inspected load ratings reveal that the Devon Bridge has undergone significant deterioration since its construction in 1905, and is due for major rehabilitation or replacement. From the six alternatives that were investigated as part of this project a shorter list of feasible rehabilitation projects should be developed that will address the deficiencies of the bridge in varying degrees.

A.) Summary of Alternatives

I. Alternative I is only an interim solution and consists of performing minor repairs or replacement of deteriorated members to increase the useful life of the bridge by 5 to 7 years. This alternative provides no operational improvements and should be implemented immediately given the likely length of time that will be needed to implement any of the other alternatives.

II. Alternative II consists of performing major repairs to both the super- and substructure, replacement of major structural members and systems, and construction of additional items to upgrade the useful life of the bridge to approximately 25 years.

IIIa. Alternative IIIa consists of replacing Spans 5, 6, and 7 of the superstructure while rehabilitating Spans 1, 2, 3 and 4, and using the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years assuming proper maintenance.

IIIb. Alternative IIIb consists of replacing the entire superstructure and retaining the majority of the existing substructure, with improvements, to increase the useful life of the bridge to 75+ years assuming proper maintenance. New movable bridge types consist of a vertical lift, Scherzer rolling lift, and heel trunnion bascule. The substructure units that support the movable span types other than the Scherzer rolling lift will require new substructure units.

IVa. & IVb Alternatives IVa & IVb consist of replacing both the superstructure and the substructure to increase the useful life of the bridge to 75+ years. The new movable bridge type for this alternative is limited to a vertical lift due to the anticipated length of the movable span. Two structure replacement alternatives were investigated: a through truss that maximizes span lengths and minimizes the number of new piers required (Alt. IVa), and a deck girder system that minimizes fracture critical non-redundant truss members (Alt IVb).

The High Towers will be replaced with monotube poles under all alternatives except Alternative I.

B.) Discussion

The existing bridge exhibits deterioration of numerous structural members, in particular stringers located below Track 1. Of particular concern is the condition of the bridge pins and eyebars on Spans 5, 6 and 7. While the pins appear to be structurally adequate, relative movement and rotation between the two components continues to wear away material in both components. The number of loading cycles has continued to increase since the bridge was constructed, and will only continue to increase based on MNR's and Amtrak's plans to increase service on the New Haven Line thereby increasing the potential for continued and increased rate of wear. The wear in the eyebars and bridge pins has resulted in numerous loose eyebars in each eyebar set, causing an uneven distribution of loads across the eyebar set that potentially can overstress individual eyebars, leading to permanent deformations. Within Spans 5, 6 and 7, there are 252 eyebar sets comprised of a total of 1200 individual eyebars, which are interconnected with 252 truss pins.

A schedule of short term repairs, similar to Alternative I and performed based on conditions found during current and future inspections, would primarily prolong the inevitable conclusion that the bridge requires significant rehabilitation or replacement to correct deficiencies that have been identified as part of this project as well as anticipated in the future.

After completion of Alternative I repairs, one of the remaining alternatives needs to be completed to keep the bridge in safe operating condition. As such, the development of construction documents and acquisition of necessary permits and approvals for the rehabilitation or replacement under Alternatives II, IIIa, IIIb, IVa or IVb should follow implementation of repairs under Alternative I. The repair program identified in Alternative I should be pursued regardless of the alternative chosen for longer term rehabilitation or replacement. This will allow the bridge condition to be maintained at a minimum until the longer term solution is affected.

A seismic analysis indicates that global pier instability (overturning) occurs at the Ultimate and Survivability (200 to 500 year and 1,000 to 2,400 year returns respectively) earthquake limit states. With any of the alternatives that retain the existing piers, the bridge will be seismically inadequate unless provisions are made to strengthen of the piers such as encasing the piers in concrete. Such work will require construction of a cofferdam to fully encase the piers for proper seismic performance. Even so, portions of the pier below grade will not be able to be encased. Alternatives IVa and IVb, as full replacements, will be the only alternatives that fully address the seismic requirements of AREMA.

A qualitative relative comparison of each alternative is listed in the following table:

Qualitative Comparison of Alternatives						
Alternative:	I	II	IIIa	IIIb	IVa	IVb
Construction Duration	+		-	-	-	-
Constructability	-	-			+	+
Operational Impacts to Rail Traffic	+		-	-	-	-
Operational Impacts to Marine Traffic	+	+	+			
Reliability	-			+	+	+
Fracture Critical Members	-	-	-	-	-	+
Seismic Performance	-	-	-	-	+	+
Elimination of Pin Connections	-	-	+	+	+	+
Environmental Impacts	+					
Corrects High Tower Deficiencies	-	+	+	+	+	+
Historic Impacts	+	+			-	-
Maintenance	-	-	-		+	+
Initial Cost	+				-	-
Annualized Cost	N/A	+			-	

Legend:
+ = Comparative Advantage
- = Comparative Disadvantage
[blank] = Negligible Comparative Advantage/Disadvantage

Several important points that should be noted when considering the advantages and disadvantages of the alternatives:

- Only Alternatives IIIa, IIIb, IVa, and IVb address the previously noted concerns with the pin and eyebar connections of Spans 5, 6 and 7. Fracture critical members will remain for all alternatives: tension members of the trusses for Alternatives I thru IVa, and the bottom flanges of the two girder deck system of Alternative IVb;
- Alternatives IIIb, IVa, and IVb will facilitate higher allowable speeds across the bridge due to the entirely new superstructure. If coupled with other improvements on the New Haven Line, will result in a time savings for passengers. User cost benefits associated with these improvements are not included in this analysis, as other improvements outside the project limits are necessary for the higher speeds on the MNR New Haven Line/Amtrak NE Corridor to be realized;
- The annualized long term maintenance costs for Alternatives II and IIIa are significantly higher than those of Alternatives IIIb, IVa, and IVb, as existing superstructure components are retained;
- Alternatives I and II offer considerably less construction impacts to rail operations due to the shorter construction duration. In particular, Alternative I repairs will have minimal impacts to rail due to the nature of the repairs that will be performed. Alternatives IIIa through IVb will significantly impact operations for up to four years while the work on the bridge is being completed;
- Alternatives IVa and IVb will result in the largest amount of long term environmental impacts due to the installation of the new piers in the river. Alternatives IIIa and IIIb also present long term environmental impacts due to the encasement of the existing piers in concrete. Alternatives II through IIIa will also have possible short term environmental impacts due to the cleaning and painting of the existing structure;
- Because the bridge consists of two largely independent structures with work generally isolated between two stages, it is possible for the entire project to be staged to match available funding. While this is not recommended due to increased mobilization and other procurement costs, Stage 1 work could be bid as one project, with the Stage 2 work bid at a later date. Further investigation sequencing the funding in this manner would be required to determine possible conflicts, overlap, and timing issues between the stages. For example when considering Alternatives IVa and IVb, the waterway area will be restricted to a minimum after the cofferdam and piers of Stage 1 are installed. The hydraulic adequacy of this condition will need to be investigated based on the time period anticipated before the existing piers are removed and the cofferdams removed; and
- Alternative IVa and IVb will allow for a much larger navigation channel due to the increased length of the movable span of the Devon and future pier locations of the Moses Wheeler Bridge.

Specific advantages and disadvantages of each alternative are listed in the following page.

Alternative Advantages and Disadvantages		
Alternative	Advantages	Disadvantages
I – Short Term Repair	• Lowest initial construction cost • Least impacts to rail operations during construction • Least impacts to marine operations during construction • Least impacts to adjacent station operations during construction • Shortest construction duration • Lowest environmental impacts • Bridge retains historical value • Lowest construction cost	• Additional work required in 5-7 years upon completion and thereafter • Labor intensive construction methods required • Continued on-going maintenance and operation cost associated with bridge • No seismic retrofit to address vulnerability to seismic events • Does not address High Tower inadequacies • Piers remain susceptible to scour • Retains fracture critical pin and eyebar connections at Spans 5, 6 & 7 • Speed of rail traffic will remain slow, with the lowest reliability of bridge performance
II – Rehabilitation	• Low initial cost • Low annualized cost • Second least impacts to marine operations during construction • Bridge retains historical value	• Additional rehabilitation required in 25 years • Long term disruption to rail operations • Labor intensive construction methods required • Continued on-going maintenance and operation cost associated with bridge • Environmental impacts due to painting • No seismic retrofit to address vulnerability to seismic events • Retains fracture critical pin and eyebar connections at Spans 5, 6 & 7 • Requires installation of scour monitoring device or scour countermeasures • Speed of rail traffic will remain slow
IIIa – Partial Superstructure Replacement	• Less impacts to marine operations during construction • Construction duration on-site can be minimized presuming new trusses are constructed off site and floated into place • Eliminates fracture critical pin and and eyebar members • Existing bridge appearance and configuration can be replicated to some extent	• Additional rehabilitation of Spans 1-4 required in 25 years • Extensive retrofit to substructures required for seismic compliance • Continued on-going maintenance and operation cost associated with bridge • Environmental impacts due to painting • Historic impacts due to partial replacement of existing superstructure and modifications to substructure • Modifications to existing piers will increase projected waterway area, possibly adversely affecting flood elevations • Requires installation of scour monitoring device or scour countermeasures
IIIb – Full Superstructure Replacement	• Increased reliability of operations due to new movable span • Construction duration can be minimized if new trusses are constructed off site • Existing bridge appearance and configuration can be replicated to some extent • Eliminates fracture critical pin and and eyebar members	• Extensive retrofit to substructures required for seismic compliance • Historic impacts due to replacement of existing superstructure and modifications to substructure • Modifications to existing piers will increase projected waterway area, possibly adversely affecting flood elevations • Requires installation of scour monitoring device or scour countermeasures
IVa – Complete Structure Replacement with Trusses	• New structure with 75+ year life span • Increased reliability of operations due to new movable span • Minimal future maintenance costs • Seismically adequate • Foundation designed for vessel collision • Reduced overall pier width will increase waterway opening and improve river hydraulics • Somewhat replicates appearance of existing truss structure • Can accommodate high speed rail service	• Highest initial cost • Highest annualized cost • Environmental impacts associated with new piers • Historic impacts due to replacement of existing structure • Retains fracture critical and non-redundant truss system for entire bridge
IVb – Complete Structure Replacement with Deck Girders	• New structure with 75+ year life span • Lesser initial cost of the two complete structure replacement alternatives • Lesser annualized cost of the two complete structure replacement alternatives • Increased reliability of operations due to new movable span • Minimal future maintenance costs • Eliminates fracture critical pin and and eyebar members • Seismically adequate • Foundation designed for vessel collision • Can accommodate high speed rail service	• Environmental impacts associated with new piers • Historic impacts due to complete change of structure type and layout • Additional piers in waterway may have adverse hydraulic effects • Visual impact of new structure may be problematic

C.) Conclusion

As previously noted, the repairs noted in Alternative I should be performed in the immediate future in anticipation of incorporating one of the remaining five alternatives.

An initial review of the alternatives was used first to eliminate the alternatives that are obviously not reasonable based on the other alternatives. Alternative IVa was eliminated during this review, as Alternative IVb accomplishes the same primary result of Alternative IVa and a lower initial and annualized cost. Alternative II was also eliminated at this point, as it does not address the concerns of the pin and eyebar connections at Spans 5, 6, and 7. Of the remaining alternatives, Alternative IIIa was next eliminated from consideration. The annualized cost for this option, which involves partial superstructure replacement, is more than that of Alternative IVb, which will result in an entirely new structure.

Based on a review the advantages and disadvantages of the two remaining alternatives (IIIb and IVb), Alternative IVb – Full Replacement with Deck Girders appears to be the most advantageous rehabilitation option. While not the least expensive option in terms of both initial cost as well as annualized cost, this option will provide an entirely new and reliable structure that will be designed entirely in accordance with current codes and standards. The annualized cost differential between these two is approximately 13%, and the initial costs of each are comparable as well. Further, a sensitivity analysis indicates that the EUAC for Alternatives IIIb and IVb will be equal when the assumed discount rate of approximately 1.5% is used in the calculations.

The work proposed with Alternative IVb will:

- Facilitate future higher speeds through the corridor which will allow for more opportunities of funding contributions from Amtrak;
- Be composed of an entirely new structure with a 75+ year life span;
- Address deficiencies of pin and eyebar connections;
- Facilitate more conventional superstructure erection techniques (ie crane picks);
- Be seismically adequate;
- Be designed for scour;
- Be designed for vessel impact;
- Improve reliability of the structure and movable span;
- Provide for minimal future maintenance;
- Allow for increased navigation channel width; and
- Be consistent with current efforts to upgrade the Northeast Corridor to a high speed rail facility.